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REVIEW OF CONSOLIDATION GROUTING OF ROCK MASSES AND METHODS FOR EVALUATION

by

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Factors involved in the successful consolidation grouting of rock masses by cementitious grouting are reviewed. Properties of cementitious grouts, rock mass properties, grouting techniques, and methods to evaluate and to monitor the grouting job are discussed. Three cases histories of consolidation grouting performed on a rock mass for remedial purposes are presented.

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PREFACE

The study herein was sponsored by Headquarters, US Army Corps of Engineers HQUSACE, under the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program, Work Unit 32276, "Grouting Practices for Repair and Rehabilitation of Rock Foundations." The HQUSACE Overview Committee consisted of Mr. James E. Crews and Dr. Tony C. Liu. Coordinator for the Directorate of Research and Development was Mr. Jesse A. Pfeiffer, Jr. The REMR Program Manager was Mr. William F. McCleese, Concrete Technology Division, Structures Laboratory, US Army Engineer Waterways Experiment Station (WES). Mr. R. Morgan Dickinson, Rock Mechanics Application Group (RMAG), Engineering Geology and Rock Mechanics Division (EGRMD), Geotechnical Laboratory (GL), WES, prepared the report. The work was performed during the period February 1983 through November 1984. The HQUSACE Technical Monitor for the work unit was Mr. Ben Kelly. Final revisions to the report were made by Mr. Perry A. Taylor, EGRMD, who was the Principal Investigator. The report was edited by Mrs. Joyce H. Walker, Information Products Division, Information Technology Laboratory.

The study was under the direct supervision of Mr. Jerry S. Huie, Chief, RMAG, who was the Problem Area Leader, and Dr. Don C. Banks, Chief, EGRMD; and under the overall supervision of Dr. William F. Marcuson III, Chief, GL, WES.

COL Dwayne G. Lee, CE, is Commander and Director of WES. Dr. Robert W. Whalin is Technical Director.

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CONVERSION FACTORS, NON-SI TO SI (METRIC)
UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
acres	4,046.873	square metres
acre-feet	1,233.489	cubic metres
bars per metre	1,000.0	pascals per centimetre
cubic feet	0.02831685	cubic metres
cubic feet per minute	0.02831685	cubic metres per minute
cubic feet per second	0.02831685	cubic metres per second
degrees (angle)	0.01745329	radians
feet	0.3048	metres
feet per minute	0.3048	metres per minute
feet per second	0.3048	metres per second
gallons	3.785412	cubic decimetres
gallons per minute	0.00006309	metres per second
gallons per minute per pounds per square inch	0.5490276	cubic metre per minute per kilopascal
gallons per foot per minute	12.41933	cubic metre per metre per minute
inches	2.54	centimetres
miles (US statute)	1.609347	kilometres
pounds (force) per square foot	47.88026	pascals
pounds per square inch	6.894757	kilopascals
pounds (force) per square inch per inch	2.7144712	kilopascals per centimetre
pounds (force) per square inch per foot	22.620574	kilopascals per metre
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
square miles	2.589998	square kilometres

REVIEW OF CONSOLIDATION GROUTING OF ROCK MASSES AND METHODS FOR EVALUATION

PART I: INTRODUCTION

Purpose

1. This study was undertaken to review consolidation grouting methods and to look at the application of consolidation grouting as a remedial measure for structures in the US Army Corps of Engineers (CE). Consolidation grouting may be defined as the injection of grout into discontinuities in a rock mass to control fundamental rock mass properties such as permeability, strength, and deformability. Consolidation grouting applications to improve engineering properties of rock mass foundations will increase in number and scope as facilities age and deteriorate. However, modern methodologies to improve economy, reliability, and planning of grouting programs for repair and rehabilitation of CE structures are required. This report reexamines factors affecting the grouting program quality, geologically imposed conditions, grouting methods, and methods to monitor and to evaluate the quality of the grouting program. The report also reviews past consolidation grouting projects and describes the execution, performance, and evaluation of the programs. The study gives an improved awareness of the factors impacting a successful grouting program.

Scope

2. The study was limited to cementitious grouts injected into a rock mass. Three case histories are presented--the only remedial consolidation grouting case histories reported by CE. The three case histories are for the Savage River Dam Spillway, Little Goose Lock and Dam, and John Day Lock and Dam. With only a limited number of case histories available, the study focus becomes more general in nature and addresses some of the factors that have to be considered in the design, execution, and evaluation of a grouting program.

Approach

3. A comprehensive literature review was conducted to examine geological factors, grouting methods, and techniques for monitoring and evaluating a grout program. Each CE district was contacted in a search for possible case histories. It was found that, although the CE frequently performs remedial seepage control grouting, remedial consolidation grouting has been reported for only three projects. Many of the findings of the literature review can be applied to cementitious grouting in general. The material identified in the literature review and the case histories are evaluated and summarized in this report.

PART II: GEOLOGICAL ASPECTS

Factors Affecting Grouting

4. Successful injection of cementitious grout into a rock mass is governed by geological factors. When considering consolidation grouting as a remedial measure, a determination must be made of whether or not the rock mass is groutable. Groutability, as defined by Mayer (1963) and Burgin (1979), is the ability of a rock mass to accept grout and is dependent on site-specific geological features, properties of the grout mix, and injection procedures.

5. The objective of consolidation grouting is to fill the fractures or discontinuities in a rock mass with grout. The discontinuities in a rock mass control fundamental rock mass properties such as permeability, strength, and deformability. The nature of the discontinuities is a major factor in the grouting operation. An understanding of general geological principles and site-specific geology is therefore required to successfully conduct a grouting program.

6. Frequency, trace length, and orientation are the most commonly measured geometric properties of discontinuities, according to Merritt and Baecher (1981). Schwartz (1983) notes that a single value does not exist for parameters such as spacing and orientation. A distribution of values is more likely. Therefore, statistical and probabilistic methods are powerful tools for characterizing discontinuities in a rock mass. However, Schwartz (1983) also indicates that the application of probabilistic and statistical methods to rock mechanics is in its infancy.

7. Accordingly, more attention has been directed in recent years to geostatistical methods to characterize a rock mass. LaPointe (1980) applied geostatistics to site characterization in Lannon, Wisconsin, for energy storage magnets. Thorpe (1981) described with statistical methods the joint systems in the Stripa Mine, Sweden. Baecher and Lanney (1978), Baecher (1983), Schwartz (1983), and Barton (1978) have illustrated statistical methods in analyzing persistence or trace lengths of discontinuities, rock mass deformability, and other aspects of site characterization. Priest and Hudson (1976) and Hudson and Priest (1979) showed statistical distributions of discontinuity frequency and related frequency to RQD (Rock Quality Designation). RQD, or modified core recover, was developed as a means of describing the condition of

the rock mass from core borings. The RQD is obtained by measuring the cumulative (total) length of intact NX core pieces 4 in.* long or longer and dividing by the sampling length.

Discontinuity Patterns

8. Joint patterns can be associated with rock lithology in a general fashion (US Army Engineer District, Portland (1983a)). Jointing in crystalline rocks (generally intrusive igneous and metamorphic rocks) usually occurs in three principal joint set orientations. Near-horizontal sheeting joints predominate near the surface. Other joint sets are composed of a near-vertical plane and corresponding plane normal to each other. Joints in metamorphic rocks are commonly oriented at right angles with cleavage planes. Highly complex jointing can also occur, with several sets at nonright angles. Volcanics or extrusive igneous rocks are more difficult to characterize. Columnar jointing is common in basalt. These columns may have three to six sides. Pumice, scoria, tuffs, and agglomerates are also volcanic rocks. The presence of gas cavities and pipe vesicles, along with scoriaceous flow contracts, can absorb tremendous quantities of grout. The inherent, localized variability of a volcanic mass is difficult to characterize geotechnically.

9. Soluble rocks, including limestone, gypsum, and dolomites, usually contain two or three joint sets. Solutioning and subsidence require additional design considerations in grouting. Cavities, joints, and bedding planes are often clay-filled, thereby limiting grout penetration. Clastic sedimentary rocks include sandstones and shales. These rock masses generally contain three, mutually orthogonal joint sets, with one set parallel to bedding.

Discontinuity Geometry

10. Sinclair (1972) discusses some of the major factors affecting foundation groutability. The primary geological characteristic affecting grout take is the geometry of the discontinuities (or voids) to be grouted. The geometry of the discontinuities is defined by width, orientation, trace length

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* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 4.

(or length of the crack perpendicular to the joint width), surface characteristics (roughness), and filling.

11. Discontinuity width (aperture) is one of the more important factors in cement grouting. If the discontinuity is not wide enough, the cement particles will not enter. Table 1 lists minimum discontinuity widths for effective cement grouting, as discussed in the references noted.

Table 1

Minimum Apertures for Cement Grouts

Reference	Minimum Width
Kennedy (1958)	3X max particle size
Morozov and Goncharov (1970)	4X to 5X max particle size
Ruiz and Leone (1970)	0.2 to 0.4 mm
Sinclair (1972)	0.1 to 0.5 mm
Cambefort (1977)	0.15 to 0.20 mm
Burgin (1979)	0.2 mm
Houlsby (1982a)	0.5 mm
Littlejohn (1982)	0.16 mm
Bruce and Millmore (1983)	0.16 mm

12. Littlejohn (1982) states the maximum particle size of portland cement ranges between 0.044 and 0.100 mm. The apertures in Table 1 generally range from two to five times the maximum particle size for most cements. The table suggests that fractures and fissures more than 0.5 mm wide are generally groutable with portland cement grout. Shimoda and Ohmori (1982) discuss application of microfine cement, which has a maximum particle size of 0.010 mm (10 μm). Microfine cement should penetrate fractures 0.02 to 0.05 mm wide based on an aperture of two to five times the particle diameter. Discontinuities with an aperture less than the two to five times the maximum particle size are not considered groutable. The cement particles will wedge or arch across the smaller openings, prematurely blocking the smaller discontinuities.

13. Aperture is not a commonly measured discontinuity property because it is difficult to measure. Typically, aperture will vary along the length of a discontinuity. Within a borehole, aperture can be measured with impression

packers or borehole cameras. Results of aperture measurements reported by Witherspoon, et al. (1980) indicate the aperture distribution to be log normal.

14. Methods to evaluate an average discontinuity width have been devised by Snow (1968) and Bruce and Millmore (1983). Both methods are based on permeability test data.

15. Snow made a series of assumptions for his method that are listed below. Many of these assumptions were made so the Navier-Stokes equation for flow between smooth parallel plates could be applied. The Navier-Stokes equation is also known as the cubic flow law, in which flow is proportional to the aperture cubed. Snow's assumptions are these:

- a. All fluid flow is along open discontinuities.
- b. Flow in essentially parallel-walled discontinuities is laminar.
- c. Discontinuities are water saturated before testing.
- d. The rock is rigid and inert to the testing fluid.
- e. Permeabilities can be computed as though the rock were infinite and continuous.
- f. Discontinuity permeability is isotropic.
- g. The number of open discontinuities obey a Poisson distribution.

16. In the method outlined by Snow, the intrinsic permeability k is determined first in units of length-squared from permeability test data. The intrinsic permeability k can be calculated as:

$$k = \frac{CQ \ln (2L/D)}{2\pi LH}$$

where

- C = coefficient dependent on measurement units
- Q = quantity of water discharging in unit time
- $\ln$ = natural log
- L = length of borehole tested
- D = borehole diameter
- H = net head acting on the tested section

17. Secondly, the discontinuity spacing is determined. Snow recommends the pressure tests be divided into groups with similar lengths of hole tested L. Each group should contain at least 25 tests, some with a zero

discharge. The percentage of zero discharge tests within the group approaches the probability of intersecting no open discontinuities. The average number of discontinuity intersections per test length M can be determined with Figure 1. Average discontinuity spacing is then calculated by dividing the test length L by the number of intersections in a test length M .

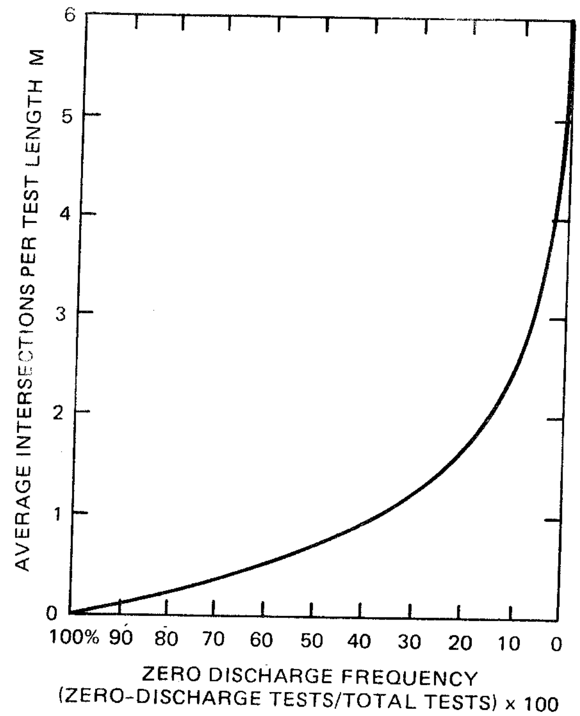


Figure 1. Average number of intersections per test, length versus probability of zero discharge frequency (after Snow 1968)

18. Snow assumed a cubical system of discontinuities, as illustrated in Figure 2. To determine the spacing between discontinuities in a set of discontinuities D , Snow introduced a modification factor. Assuming a vertical borehole drilled into the cubical system shown in Figure 2, Snow shows that

$$D = 1.25(L/M)$$

19. With a given spacing D and an intrinsic permeability k , the porosity P can be found from

$$P = 5.45(k/D^2)^{1/3}$$

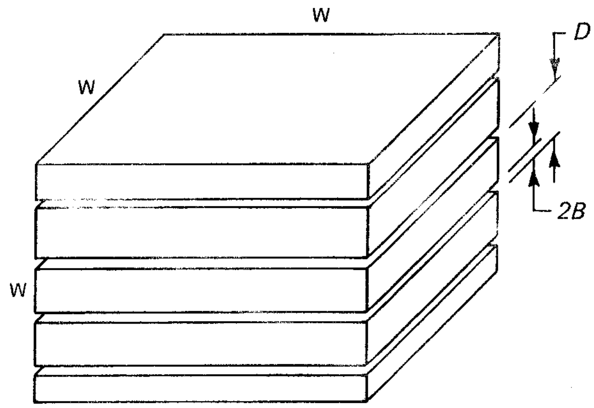


Figure 2. Cubical system of discontinuities (after Snow 1968)

for a cubical discontinuity system. Snow then shows discontinuity width 2B to be

$$2B = PD/3$$

20. Bruce and Millmore (1983) present a nomogram (Figure 3) that can be used to estimate average discontinuity width from water test data. The equation used in the development of the nomograph is for laminar, fully developed flow between two parallel plates, or the cubic flow law. The nomograph may be useful for interpretation of the water test; however, the inherent assumptions in the equation must be acknowledged. The example provided in Figure 3 illustrates the use of the nomogram.

21. Discontinuity width is affected by the applied grout pressure. Ruiz and Leone (1970) estimated that discontinuity width during grouting increased by 0.1 mm under an applied pressure of 4.0 kg/cm^2. The rock mass, a sequence of basaltic lava flows, was assumed to be a homogeneous, isotropic, elastic media. The increase in aperture is the smallest groutable fissure width, thereby increasing grout take.

22. Cambefort (1977) recommends the grout pressure be high enough to widen the discontinuity; therefore, when grouting is terminated, the elastic response of the rock will provide a tighter seal. If this recommendation is followed, care must be taken that grouting pressures are not great enough to "jack" or lift the ground surface or to cause hydraulic fracturing. In using Cambefort's recommendation, the modulus of the rock must be known to determine

EQUATION : –

$$H = \frac{6\eta}{\pi p g} \frac{Q}{nLt^3} \log_e \frac{R}{r}$$

WHERE: –

H = EXCESS HEAD AT MID-DEPTH OF SECTION BEING CONSIDERED

η = VISCOSITY OF FLUID

Q = FLOW RATE INTO HOLE OR STAGE

R = DISTANCE TO SOURCE OF WATER (EXPRESSED AS EFFECTIVE RADIAL EXTENT OF A DISC-SHAPED STRATUM

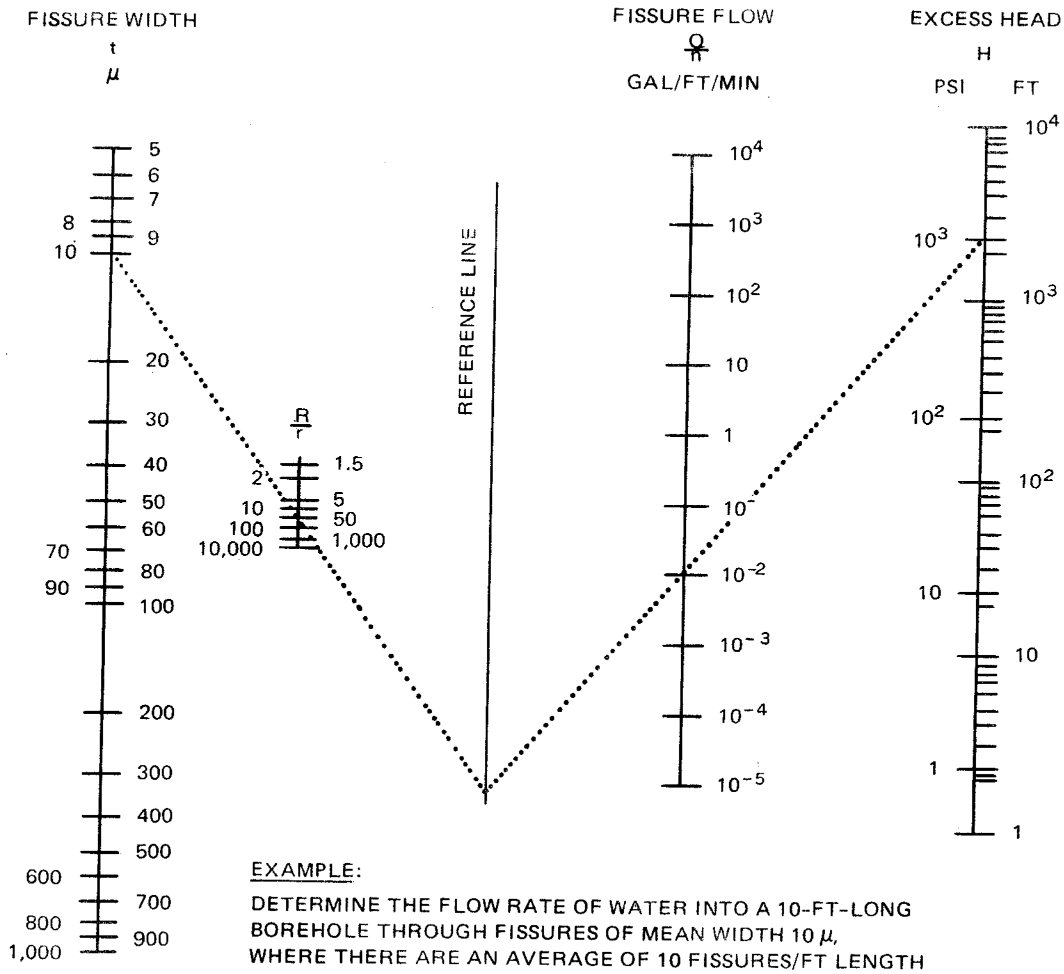
p = DENSITY OF FLUID

n = NO. OF FISSURES PER UNIT LENGTH OF HOLE

L = LENGTH OF HOLE OR STAGE UNDER TEST

t = AVERAGE FISSURE WIDTH

r = RADIUS OF HOLE



EXAMPLE:

DETERMINE THE FLOW RATE OF WATER INTO A 10-FT-LONG BOREHOLE THROUGH FISSURES OF MEAN WIDTH 10 μ, WHERE THERE ARE AN AVERAGE OF 10 FISSURES/FT LENGTH OF HOLE, UNDER AN EXCESS PRESSURE OF 1,000 PSI, THE AVERAGE VALUE OF R/r IS EQUAL TO 100.

1. PLACE A RULER ACROSS t=10 AND R/r =100 TO GIVE POINT ON REFERENCE LINE.
2. PLACE A RULER ACROSS REFERENCE POINT AND H=1,000 PSI. READ Q/n = 10^{-2}.
3. SINCE n=10 AND L=10 FT, Q=10^{-2} x 10 x 10. THEREFORE Q = 1 GAL/MIN.

Figure 3. Nomogram relating aperture and excess head to flow rate (after Bruce and Millmore 1983)

where elastic widening stops and jacking begins. Note that this procedure is not a part of CE grouting procedure.

23. An additional benefit of high grouting pressures was noted by O'Neill and Lyons (1964). Analysis of the test grouting program at Oroville Dam showed that the high grouting pressures squeezed grout into soft joint-filling material. Distinct openings in the soft material were not believed to have existed prior to grouting.

24. Discontinuity spacing has been addressed by many researchers. Priest and Hudson (1976) developed the often-referenced relationship between RQD and mean discontinuity frequency:

$$RQD = 100e^{-0.1\lambda}(0.1\lambda + 1.0)$$

where

RQD = Rock Quality Designation

e = the base of the natural log

λ = mean discontinuity frequency (fractures/metre)

This relationship is derived assuming discontinuity spacing fits a negative exponential distribution. In the study and subsequent publication (Hudson and Priest 1979, 1983), a negative exponential distribution of discontinuity spacings is shown to be a realistic assumption.

25. The orientation of discontinuities is another factor influencing grout operations. The probability of a vertical grout hole intersecting a vertical joint is low. Terzaghi (1965) developed the concept of blind zones encountered in joint surveys. The orientation of a given outcrop or borehole precludes observation of discontinuities with parallel orientations. The optimum grout hole orientation would be normal to the discontinuity to be grouted. Kreuzer and Schneider (1970) used stereographic projections to determine the optimum grout hole orientation based on this principle.

26. The discontinuity orientation in relation to the grout hole orientation also determines whether applied pressure will open or close the discontinuity in response to applied pressure. This was shown experimentally by Barroso (1970). He constructed a rock mass model consisting of lucite blocks with three orthogonal sets of fissures. Some fissures opened during pressure application to admit grout. Other fissures remained closed and were not grouted. We can envision fissures intersecting the grout hole at small angles

pressed closed by the application of grouting pressure; conversely, we can envision fissures perpendicular to the grout hole opening up as pressure increases.

27. Discontinuities are often assumed to extend areally over wide extents (Sinclair 1972). A joint system with a high degree of continuity will accept more grout than a discontinuous joint system. Sinclair points out there are no practical methods to determine continuity in the subsurface.

28. Probabilistic methods have been applied by Baecher and Lanney (1978) and Barton (1978) to discontinuity trace length. The statistical treatment indicates trace lengths fit log-normal distributions; however, Merritt and Baecher (1981) indicate some researchers have assumed exponential distributions of trace length.

29. Baecher and Lanney (1978) investigated the sources of sample bias in discontinuity trace-length surveys conducted on surface outcrops. The biases are from orientation, as shown by Terzaghi (1965), and also from size bias, truncation bias, and censoring bias. Size bias means that longer trace lengths have a greater probability of being exposed than shorter trace lengths. Truncation bias occurs when trace lengths less than a given size are not recorded because of hole spacing. Censoring bias is present when one or both ends of the discontinuity are not exposed in a surface survey. When known to exist, these biases should be accounted for during a subsurface joint survey.

Additional Geological Factors

30. Additional geological factors affecting grout take have been discussed. Houlsby (1982a) mentions rock strength, rock soundness, and the in situ stress state. Jawanski (1970) includes the degree of tectonic disturbance and weathering. Roughness and filling of the joints will influence grout take. A clean, smooth joint will readily accept grout. A rough joint surface will inhibit grout flow, as will fracture fillings. Weathering processes and tectonic disturbances (folding and faulting) will often produce clayey gouge or joint filling.

31. The in situ stress state may be altered by grouting procedures. This can result in hydrofracturing, uplift, and associated rock mass behavior

generally considered to be detrimental to a successful grouting operation. Allowable grouting pressures are discussed more fully later in this report.

32. The presence of ground water may alter proposed grouting procedures. For grout to displace ground water, the pressure driving the grout must be greater than the head acting on the ground water. If ground water is flowing with a significant velocity, the grout may be diluted or washed away before the cement sets. Walley (1976) injected a wide variety of grout mixes into a tank with flowing water. He concluded fast-setting grouts were most resistant to erosion and dilution of flowing water. Bentonite mixtures were also resistant to erosion.

33. Ground-water chemistry should be considered. The US Army Engineers (1984) recommends ground-water sample pH and chemistry be tested. A study by Gale and Reardon (1984) showed neat portland cement grout to dissolve when distilled water flowed through a grouted discontinuity; however, when a supersaturated solution of $Ca(HCO_3)_2$ was the influent, flow rates decreased markedly with time because of the calcite precipitation. This study indicates that ground-water chemistry is an important factor when considering durability of a grouting operation.

PART III: GROUTING METHODS

34. Grouting methods are fully documented in other publications. Detailed methodologies can be found in Bowen (1975), US Army Engineers (1984), and Albritton, Jackson, and Bangert (1984). The discussion presented in this part touches on some of the methodologies affecting consolidation grouting operations and quality.

Evaluation of the Project

35. The purpose of a remedial consolidation grouting program is to improve the mechanical properties of a rock mass. Cement grout injection for rock mass improvement has two inherent contradictions (Adamovich and Baushev 1970). First, effective grout penetration is facilitated with high injection pressures. To limit additional damage to fissured rock masses and also to produce a high quality grout operation, limitations on applied pressures are imposed. Second, a thin grout dilution has better penetration, but a maximum cement concentration is required to obtain substantial improvement of the rock strength and stiffness properties.

36. The individual characteristics of the rock mass to be grouted can vary greatly from grout hole to hole. A cursory examination of grout records from any job will show variations in grout take in adjacent holes. This was shown during the remedial consolidation grouting done at John Day Lock and Dam. Figure 4, from Neff, Sager, and Griffiths (1982), shows a variation in average grout take of the primary holes of 2.5 to 3.5 sacks of cement per linear foot of grout hole between adjacent monoliths 17 and 19. This observation implies that flexibility and experience are required to accurately estimate grout take for a grouting program.

37. Before deciding to embark upon a remedial consolidation grouting program, an evaluation of the economic and technical aspects of the project should be made. The site-specific problem to be solved by the grouting program is defined in a way that indicates the required depth and areal extent of grouting. Economics require that the minimum amount of grouting necessary to correct the problem should be accomplished.

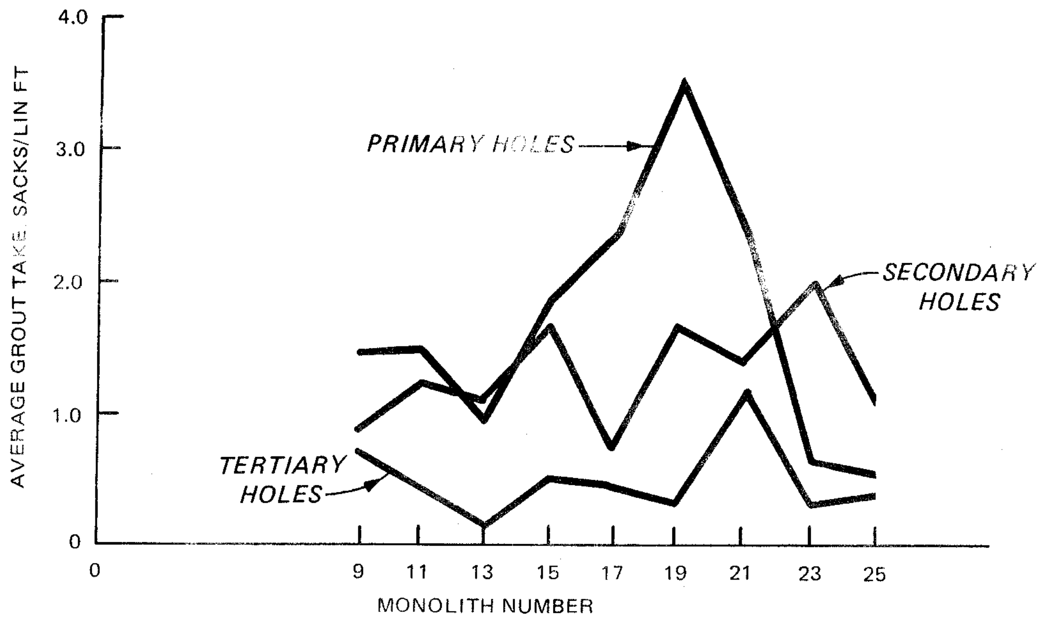


Figure 4. Average grout takes in zone 2 versus monolith location (after Neff, Sager, and Griffiths 1982)

Stage and Stop Grouting

38. Consolidation grouting is generally performed by stage or stop grouting. Stage grouting is sometimes called "downstage" while stop grouting is known as "upstage" grouting. More complete discussions of the methods can be found in US Army Engineers (1984), Bowen (1975), and Houlsby (1982a), as well as many other sources. In both stage and stop grouting, the zone to be grouted is isolated by the use of packers in stop grouting or by the bottom of grout holes in stage grouting. Each isolated zone or stage is grouted until refusal criteria (generally a limiting pressure combined with a minimum flow rate) are obtained. A thin grout mix is injected initially, and, if refusal is not obtained, the mix is gradually thickened. Figure 5 shows downstage and upstage grouting methods schematically.

Grout Hole Spacing

39. The split spacing method is often implemented to reduce the risk of incomplete grout coverage. Split spacing simply means a series of primary holes are drilled and grouted first. The distance between the primary holes

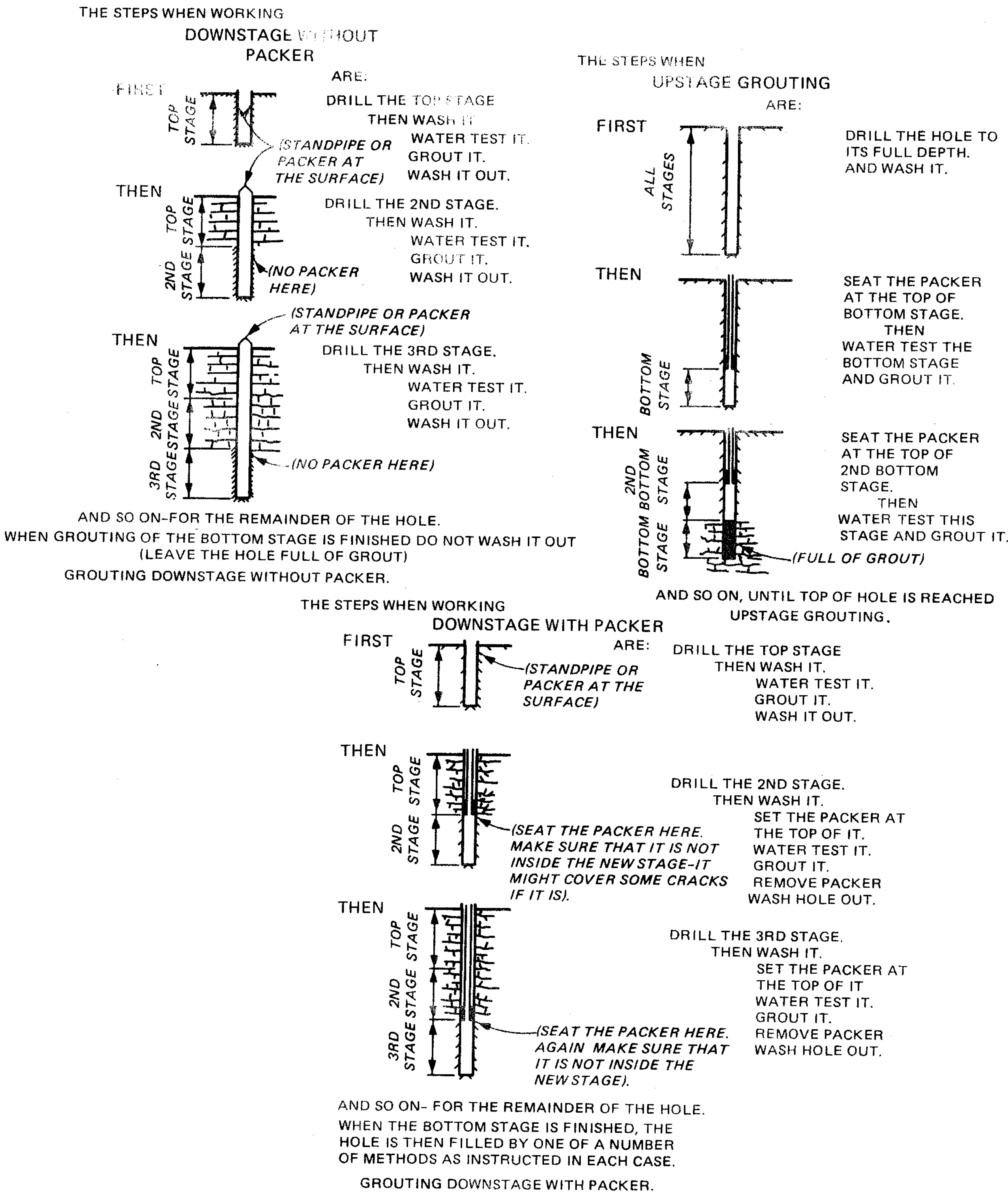


Figure 5. Grouting methods (after Houlsby 1982a)

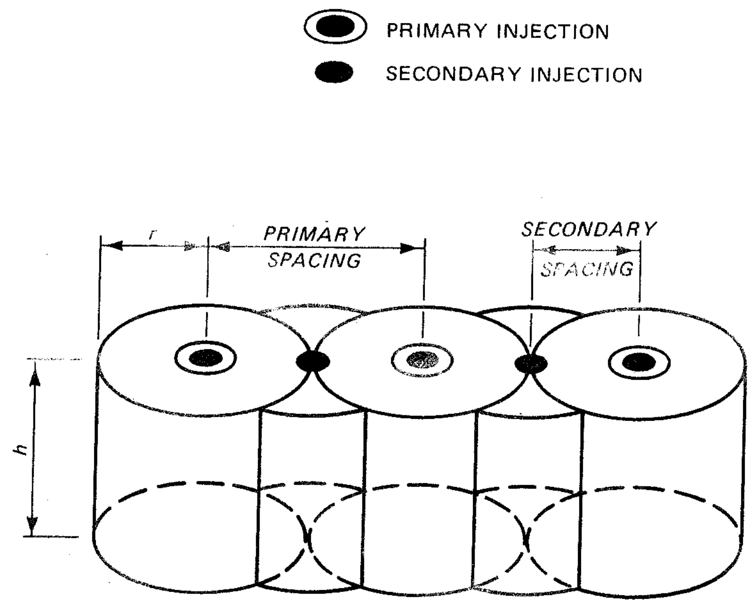
is halved, and secondary holes are drilled and grouted between the primary holes. This process can continue to include tertiary and quarternary holes, as required.

40. Spacing of primary grout holes is generally 10 to 40 ft for curtain grouting (US Army Engineers 1984). Nonveiller (1970) explains the primary spacing is dependent on fracture system characteristics, rock mass permeability, grouting techniques, and pressures. He also believes test grouting is the only practical method to determine ideal spacing. An idealized cylinder of grouted rock can be envisioned as the result of grouting one hole. Nonveiller (1970) indicates the primary spacing should be greater than the idealized cylinder diameter. Grant (1964b) presents a similar argument, drawing on the concept of productive interference. The point of productive interference corresponds to the grout hole spacing at which the adjacent injected grout masses interact. Closure grouting, usually done with split spacing, is then used to fill in areas between adjacent idealized cylinders associated with the primary holes. This concept is shown in Figure 6, after Sinclair (1972). Similar to the concept of productive interference is the concept of diminishing returns. The point of diminishing returns occurs when grout absorption decreases to the point where the net effect is backfilling the grout hole. Figure 7, after Sinclair (1972), shows the relationship between hole spacing and grout take along with the concepts of the point of productive interference, closure grouting, and the point of diminishing returns.

41. Fergusson and Lancaster-Jones (1964) defined a reduction ratio (also shown in Figure 6) which is primarily a method to evaluate the grouting job. The reduction ratio is defined as the ratio of the average grout take of a particular phase to that of the previous phase (i.e., secondary to primary). The average grout take is expressed as the volume of grout injected per linear foot of hole.

Grout Hole Drilling

42. The grout holes should be drilled in direction and angle to intersect as many discontinuities as possible. If the primary set of discontinuities to be grouted consists of horizontal sheeting joints, for example, the ideal hole orientation would be vertical, normal to the set of discontinuities



ASSUME h = r = 1.0

UNIT VOLUME, PRIMARY SPACING = 3.14

UNIT VOLUME, SECONDARY SPACING = 0.68

REDUCTION RATIO = $\frac{0.68}{3.14}$ = 0.22

Figure 6. Idealized grout cylinders and closure (after Sinclair 1972)

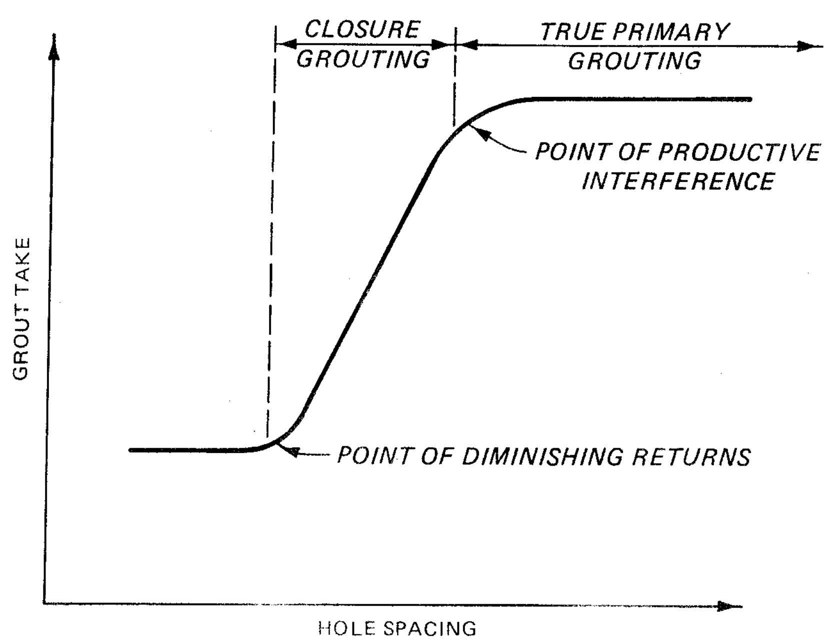


Figure 7. Theoretical relationship between grout take and hole spacing (after Sinclair 1972)

to be grouted. Kreuzer and Schneider (1970) developed a method based on stereographic projections to determine the optimal grout hole orientation.

43. Grout hole drilling is the most cost-intensive item of a group program. In Grant's (1964a) analyses of three TVA dams, he found, on the average, 1.4 lin ft of drilling was required to place 1 ft^3 of grout. Table 2 was prepared from data presented by Albritton, Jackson, and Bangert (1984). The cost of drilling ranges from 13 to 84 percent of the total grouting program cost. The overall average drilling cost from the available data is about 52 percent (over one-half) of the entire grouting program cost.

Table 2

Comparative Drilling and Grouting Costs

Dam Project	Drilling Costs, $	Total Grout Job Costs, $	Ratio, Percent Drilling/Total Costs, $
Norfork	191,174	227,861	84
Oolugah	13,659	26,005	53
Alvin Bush (1958)	20,371	159,328	13
Alvin Bush (1964)	136,476	236,205	58
Alvin Bush (1974)	155,345	321,075	48
Abiquiu (1963)	71,332	248,722	29
Abiquiu (1967)	300,352	643,661	47
Abiquiu Inc-I (1980)	917,780	2,202,110	42
Abiquiu Inc-II (1980)	796,691	1,749,819	46
Dworshak	649,728	835,522	78
Laurel	46,195	110,158	42
Clarence Cannon	1,418,880	2,282,647	62
Total	4,717,983	9,043,113	52

Note: Data taken from Albritton, Jackson, and Bangert (1984).

44. The grout hole is important because it serves as the connection between the grout plant and the rock mass discontinuities to be treated; therefore, drilling procedures should result in a clean hole, and the intersecting discontinuities need to be open and clean.

45. The grout hole diameter is one of the many factors to be decided in design phases. Controversy exists as to the effect of hole diameter. A test grouting program, reported by O'Neill and Lyons (1964), found varying hole diameter from EX (1-1/2-in. diam) to NX (3-in. diam) had no appreciable effects on grout take. A grouting program analyzed by Lane (1963) showed no practical difference in grout take from drilling 2-in.- or 2-1/2-in.-diam holes. However, Albritton (1982) indicated that greater grouting pressures have to be applied when small-diameter holes are drilled to achieve the same net improvement in the subsurface. Figure 8, after Cambefort (1977), shows the pressure distribution versus increasing distance from the borehole. The figure shows that with the same pressure P_o at the borehole wall, the fissure will experience a higher pressure at a given distance for the larger diameter grout hole. This will be discussed later in reference to allowable grouting pressures.

46. The US Army Engineers (1984) bases hole diameter on factors such as geology and hole inclination. Smaller diameter holes may drift but can be

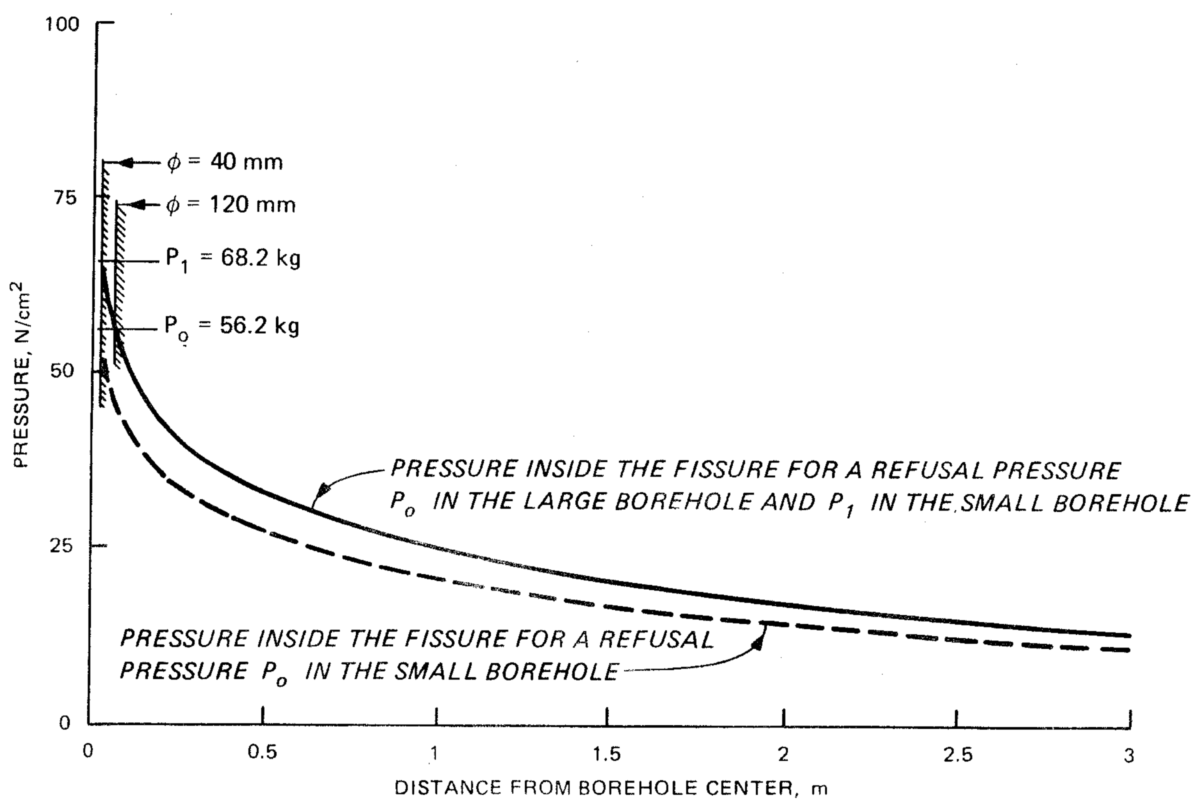


Figure 8. Distribution of pressures as a function of the distance from the hole in the case of a constant width aperture (after Cambefort 1977)

drilled easily in harder rock types. Larger holes will experience less drift because of stiffer drill rods. Albritton (1982) indicates significant drift is not expected until depths of 200 ft are obtained. Consolidation grouting is normally conducted at relatively shallow depths; therefore, grout hole drift is not expected to be a problem.

47. Percussion and rotary drilling methods are both widely used for grout holes. Discussion regarding relative benefits of each method has been continuing for many years. It is generally recognized that percussion drilling is faster and costs less; however, premature plugging of fine cracks and fissures may result from the drilling method. Bussey (1963) noted that grout take in percussion drilled holes was only 30 percent of the grout take in rotary holes drilled in diorite. Grant (1964a) showed rotary diamond core drilling based on grout take was five times better than percussion drilling methods. Grant's conclusion was based on analyses of three sites. Underlying bedrock for the three projects included shale with thin limestone beds, sandstone with shale partings, and dolomite.

48. Houlsby (1982a) points out that the fundamental issue is to select an economical drilling method that does not clog discontinuities with cuttings. If percussion drilling is selected, Houlsby believes that it must be wet drilling. The site should be tested to determine suitability of various drilling methods and how readily the cuttings can be flushed out after drilling. He also states diamond drilling is not necessarily better in all cases. Some sites have produced sharp, clean cuttings with percussion drilling, where diamond drilling has yielded dirty grout holes. Albritton (1982) showed percussion drilling with air can cause premature plugging in soft rock such as shale.

49. The US Army Engineers (1984) states, when rotary drilling, the drilling fluid pressure can force cuttings into the discontinuities. Houlsby (1982a) believes the higher air pressure can plug fissures more readily. The US Army Engineers (1984) also indicates soft rock can smear with percussion drilling and circulation loss is more difficult to observe when air is the drilling fluid. The danger of a blockage in the hole, thereby allowing full air pressure to be applied to the foundation, has led the CE to prohibit drilling with air in embankments and dam foundations.

50. After drilling is completed to the desired depth, it is universally recognized that cuttings must be flushed from the hole prior to grouting.

This requirement has often been expanded to an attempt to clean the discontinuities to be grouted. O'Neill and Lyons (1964) concluded pressure washing to clean fractures does not work, and it may be undesirable as the clay fillings may be less pervious than the cement grout. However, they were grouting for seepage control and were not attempting consolidation grouting. Mayer (1963) felt that washing is essential to a successful grouting program; however, he noted that clay fillings cannot always be washed out. Lane (1963) also commented on pressure-washing techniques--the reported project met limited success. Some of the clay was removed, and connection was observed between grout holes; however, a postgrout examination of the grouted rock mass showed that not all of the clay had been removed from the fractures.

Grouting Pressures

51. To obtain maximum grout penetration, maximum pressures are required; however, applying too much pressure can cause irreparable foundation damage, i.e., uplift and fracturing. The US Army Engineers (1984) presents a "rule-of-thumb" grouting pressure criterion of 1 psi/ft of depth in rock, and 0.5 psi/ft of depth in overburden soils. The reasoning behind application of this criterion is to limit the pressure to overburden weight to prevent uplift. On the other hand, Albritton, Jackson, and Bangert (1984) point out the growing recognition that higher pressures can be applied safely.

52. The maximum grouting pressure is dependent on several factors in addition to overburden weight. Morgenstern and Vaughn (1963) state the pressure is dependent on rock strength, in situ stresses, and existing pore pressure in the rock. Sinclair (1972) points out rate of grout acceptance, geologic structure, grout consistency, and previous grouting, if any, should also be considered. Sinclair (1972) discusses a concept of uplift resistance of a rock mass. The resistance is dependent on rock strength, degree of jointing, and in situ stress state and is influenced by grout consistency and flow rate. As the areal extent of the pressurized grout increases, the uplift resistance is reduced. Also, as the pressure acts on increasing areas, the risk of uplift increases with time; however, he does not present a methodology to determine uplift resistance. Houlsby (1982a) suggests increasing pressure should be applied slowly to allow assessment of the effects. Soejima and Shidomoto (1970) indicate foundation damage is primarily related to flow rate

and pressure. O'Neill and Lyons (1964) point out pressure surges, as related to intensifier-type pumps, can increase the risk of uplift. A range of allowable pressures implemented in the past is shown in Figure 9, after Houlsby (1982a). Houlsby notes that, in general, European practice allows a pressure gradient of 1 bar/m of depth (about 4.5 psi/ft); whereas American practice is 1 psi/ft. Figure 9 graphically indicates the difference between the allowable gradients. Houlsby shows how the pressure gradients can pertain to soundness of the rock mass. As shown in Figure 9, lower pressures should be applied to weak or highly fractured rock. Sound, strong rock will accept higher pressures before damage occurs.

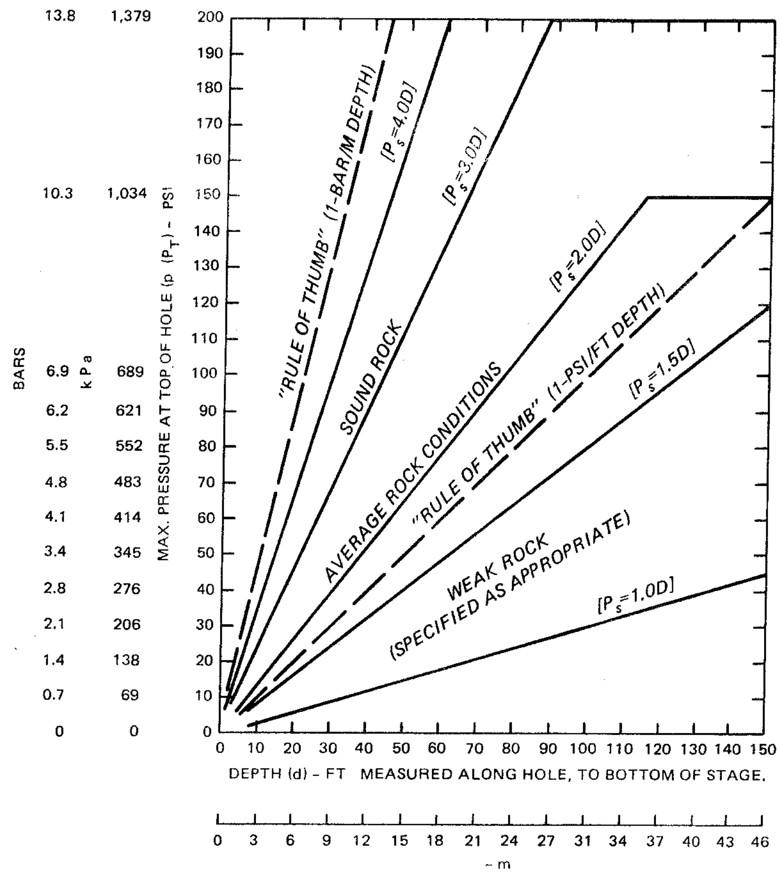


Figure 9. Recommended maximum grout pressures (after Houlsby 1982a)

53. The relative importance of foundation damage can be deduced from the laboratory shear strength tests conducted by Deere and Coulson (1971). They found the shear strength of a natural rock joint is greater than a grouted rock joint, and the strength decreased as the thickness of grout increased. The strength decrease was attributed to loss of contact of surface

asperities. Up to a limiting grout thickness, the asperities would still have a degree of interlocking. As grout thickness increases, the interlocking decreases until there is no intersection of the asperities during shearing. The shear strength is then equal to the grout strength.

54. A jointed rock model grouted by Barroso (1970) indicated the difference in grout penetration obtained by varying the rate of pressure application. When maximum pressure was applied at the beginning of the grout operation, the fissures opened immediately to accept the grout, and better penetration of all fissures was achieved than when pressure was applied incrementally. This is at variance with Houlsby's (1982a) recommendation of slowly applying the pressure; however, his intent was to minimize foundation damage and not to maximize grout take.

55. Bruce and Millmore (1983) used hydrofracturing to estimate the maximum allowable pressures a foundation could accept. This may not be applicable for remedial grouting because of existing structures and possible damage to them.

56. Morgenstern and Vaughn (1963) and Sinclair (1972) point out the grouting pressure will open the fissure to be grouted. The widening may be about 0.1 mm, the minimum groutable width for cementitious grout based on particulate diameters. This aspect of grouting was discussed earlier.

57. In calculating the pressure at the point of injection, Sinclair (1972) shows the weight of the grout column and water pressure in the rock should be considered. The pressure at the point of injection is not the gage pressure measured at the collar.

Grout Mixture and Equipment

58. Typically, during the grouting operation, the grout mixture is thinned or thickened in response to conditions encountered. Each hole is generally started with a thin mixture. If refusal is not obtained in a reasonable period, the grout is thickened. This procedure is justified by O'Neill and Lyons (1964). They believe thin grout, with better penetrating ability, will fill the small cracks and fissures. The thicker grouts will then fill the larger connecting fissures. In the United States, the water-cement ratio of the initial mix generally ranges between 3:1 and 5:1. Water-cement ratio, unless otherwise specified, is the ratio of water, in ft^3, to sacks of cement,

given as 1 ft^3, loose volume of cement. Houlsby (1982b) recommends beginning with a 2:1 mix, based on experience and demonstrated durability.

59. Grouting equipment considerations are fully discussed in other publications. Some of the more important concerns, as they apply to consolidation grouting, include the mixer and grout pump.

60. One of the most influential equipment items is the mixer. The high-speed "colloidal" mixers produce a grout with higher fluidity and greater stability than paddle-type mixers, as shown by Mayer (1963), O'Neill and Lyons (1964), Burgin (1979), and Albritton, Jackson, and Bangert (1984). It is believed high-speed mixing breaks up particle agglomerations, thereby increasing uniformity of the grout mix. High-speed mixers may also reduce cement particle size by rounding the sharp, angular pieces.

61. Grout pumps also need to be considered. Piston-type or intensifier pumps can produce a pressure surge on stroke reversal. The surge may cause fracturing or uplift of the foundation. The Moyno or screw-type pumps do not exhibit the surging action and are preferred by most practitioners.

PART IV: MONITORING AND EVALUATION OF GROUTING

62. A successful consolidation grouting program results in an increased strength and stiffness and a reduced permeability of a rock foundation. This is accomplished by filling voids and fissures in the rock mass with grout, and displacing the air, water, and, on occasion, clay or other infillings. Kujundzic (1966) states:

> By filling the fissures and cavities in a rock mass with cement grout, its individual parts become connected and thus its compactness increases. In this way, instead of a discontinuous medium, partial displacements of individual rock parts are eliminated, and so the deformability of the rock mass is, in fact, reduced and the value of the deformation modulus increased.
>
> Besides, the marked reduction of deformability by consolidation grouting we succeed in making homogeneous rock masses that are heterogeneous by their parameter of fissurability. Individual zones, fractured in a varying degree and having therefore various deformation moduli, acquire through grouting such values of deformation moduli that they approach each other. This applies also to the reduction of the degree of anisotropy when the anisotropy, by the parameter of deformability, is provoked by the predetermined or existing directions of the fissure systems.

63. To perform a successful grouting job, the grouting operation must be monitored and supervised in the field by a competent engineer or geologist. The experience of field personnel often determines the quality of the grouting job.

64. A postgrouting evaluation of the job allows an assessment of the adequacy of the job. All too often, a postgrouting evaluation consists of observing structure performance. More recently, geophysical testing has been conducted before and after grouting to evaluate the degree of grouting effectiveness.

Electronic Monitoring

65. Recent application of microcomputer technology is changing traditional monitoring methods. Field monitoring typically consisted of recording pressure, mix ratios, and quantity of grout pumped. The grout quantity was measured by recording the height of grout remaining in the mixing or holding

tank at specific time intervals. Now, equipment is commercially available to record highly accurate pressure flow rate and flow quantity readings in real time. Davidson (1984) discusses the grout monitoring program recently conducted by the US Bureau of Reclamation. Jefferies, Rogers, and Reades (1982) discuss a similar application and the associated productivity gains possible.

66. These monitoring systems produce real-time plots of pressure and flow rate. Figure 10 shows typical plots from Jefferies, Rogers, and Reades (1982). Figure 11, from Davidson (1984), shows some of the variations available. By watching the flow versus time and pressure versus time plots, experienced inspectors at the grout hole determined refusal and identified leaks and other problems. Once the system's "bugs" had been worked out, the Bureau inspectors felt the system worked well and improved efficiency and job quality.

67. Another aspect of electronic monitoring is the capability of monitoring several holes simultaneously. Mueller (1982) and Davidson (1984) report on multiple hole hookups. In both reported cases, equipment allowed five grout hookups and one water pressure test connection.

68. In the case of electronically monitored grouting, equipment downtime becomes important as in any technology-based operation. However, in the cases discussed previously, the problems were solved. Davidson states these are the primary problems encountered:

a. Line voltage variations.

b. Microcomputer malfunctions, primarily dust related.

c. Software problems, primarily loss of incoming data while plotting.

d. Personnel resistance to change in standard operating procedures.

69. The equipment is not meant to completely replace experienced personnel. Experience is still required to interpret the data, to determine mix change, when to stop grouting, and so forth. The equipment allows faster interpretation and more comprehensive analysis. In short, microcomputer technology is a tool that can increase productivity, job quality, and reduce costs when properly applied.

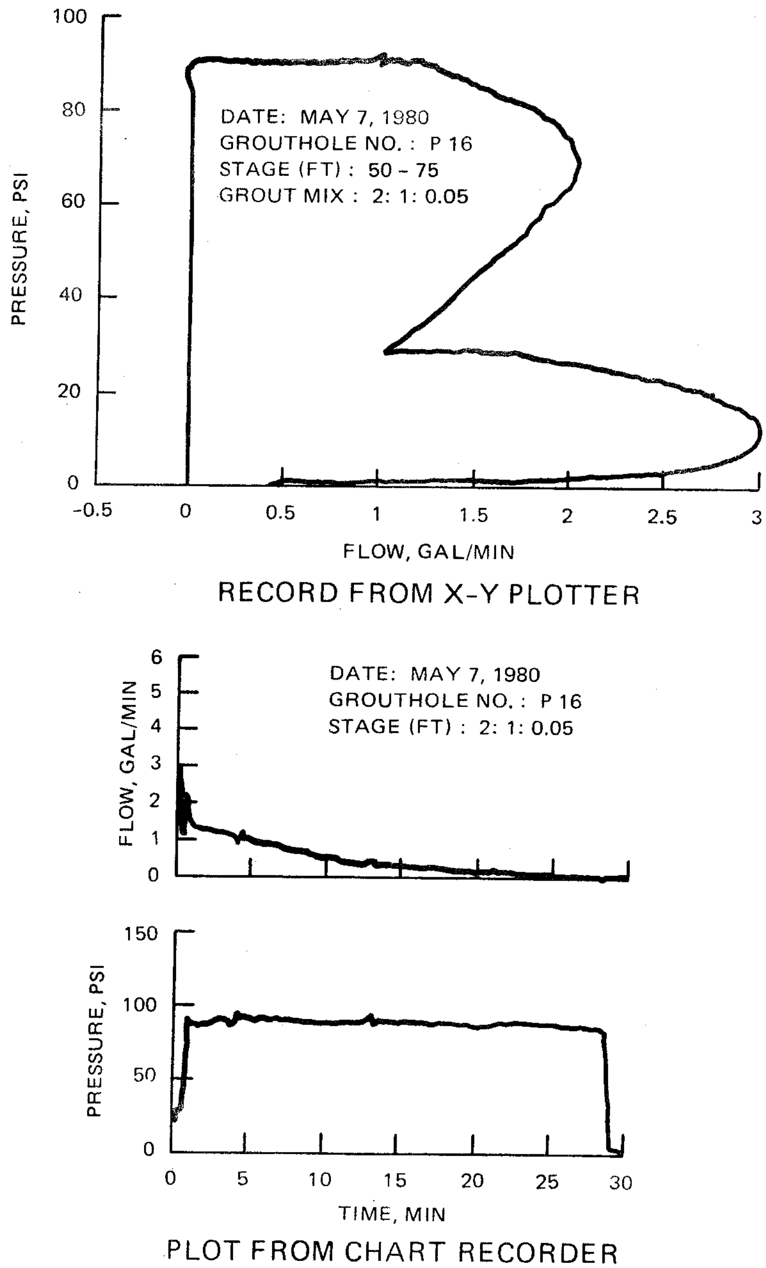


Figure 10. Typical grout injection records (after Jefferies, Rogers, and Reades 1982)

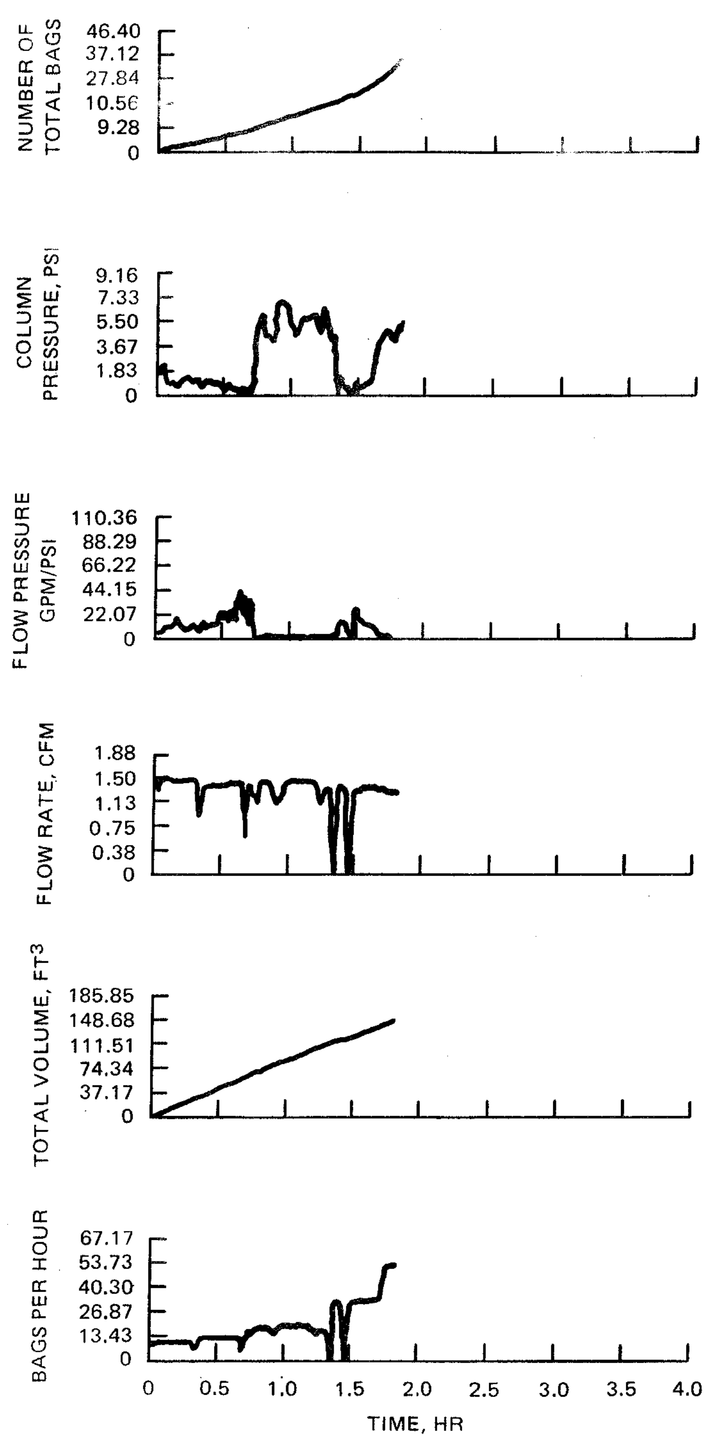


Figure 11. Typical hardcopy plots drawn after each stage of grouting (after Davidson 1984)

Effects of Grouting

70. Improvement of the mechanical rock mass characteristics by grouting has been noted by several investigators. Barroso (1970) found the rock-grout shear strength to be greater than rock-rock shear strength in the Mohr-Coulomb failure criterion. Most of the strength gain was from the addition of a cohesion component in the Mohr-Coulomb failure criterion. An increase in the friction angle d was noted for the rock grout interface on smooth, sawed rock surfaces. Natural joint surfaces showed very slight increases in strength for the rock-grout interface. Barroso (1970) did indicate a detrimental effect on strength from the loss of asperity contact. The strength of the joint, when asperities did not influence strength, was equal to the strength of the grout.

71. Deere and Coulson (1971) concluded grouting does not improve the shear strength of the joint. At low normal stresses, the cohesion component of shear strength contributed to an increase in total shear strength; however, grouted joints had a reduced friction angle, as shown in Figure 12. At normal stresses less than 100 psi, the failure was within the grout. At normal stresses greater than 100 psi, failure occurred at the rock-grout interface.

72. Evdokimov et al. (1970) cite two field tests conducted by other investigators in the USSR showing 7- to 10-percent and 3- to 10-percent increases in strength, respectively. To further investigate these findings, a laboratory test program was initiated. Natural joints were sheared in the laboratory. The joints were grouted after shearing without removing resulting gouge material. This sequence was to represent a "natural" tectonic disturbance. The specimens were resheared after grouting, and strength increases of 32 to 259 percent were noted in the limiting shear strength. Residual sliding tests, in which the samples were resheared after the initial grout bond was broken, indicated a reduced friction angle as the influence of asperities was masked by the grout.

73. These series of test results were all conducted in differing fashions but seem to agree that the grout imparts cohesion to the joint; however, because the concrete is generally weaker than the rock, the friction angle ϕ is lower. Also, the grout masks or hides asperities, resulting in lower friction angles. The nature of laboratory shear testing is such that the joints are closed by application of normal stresses prior to shear. In grouting

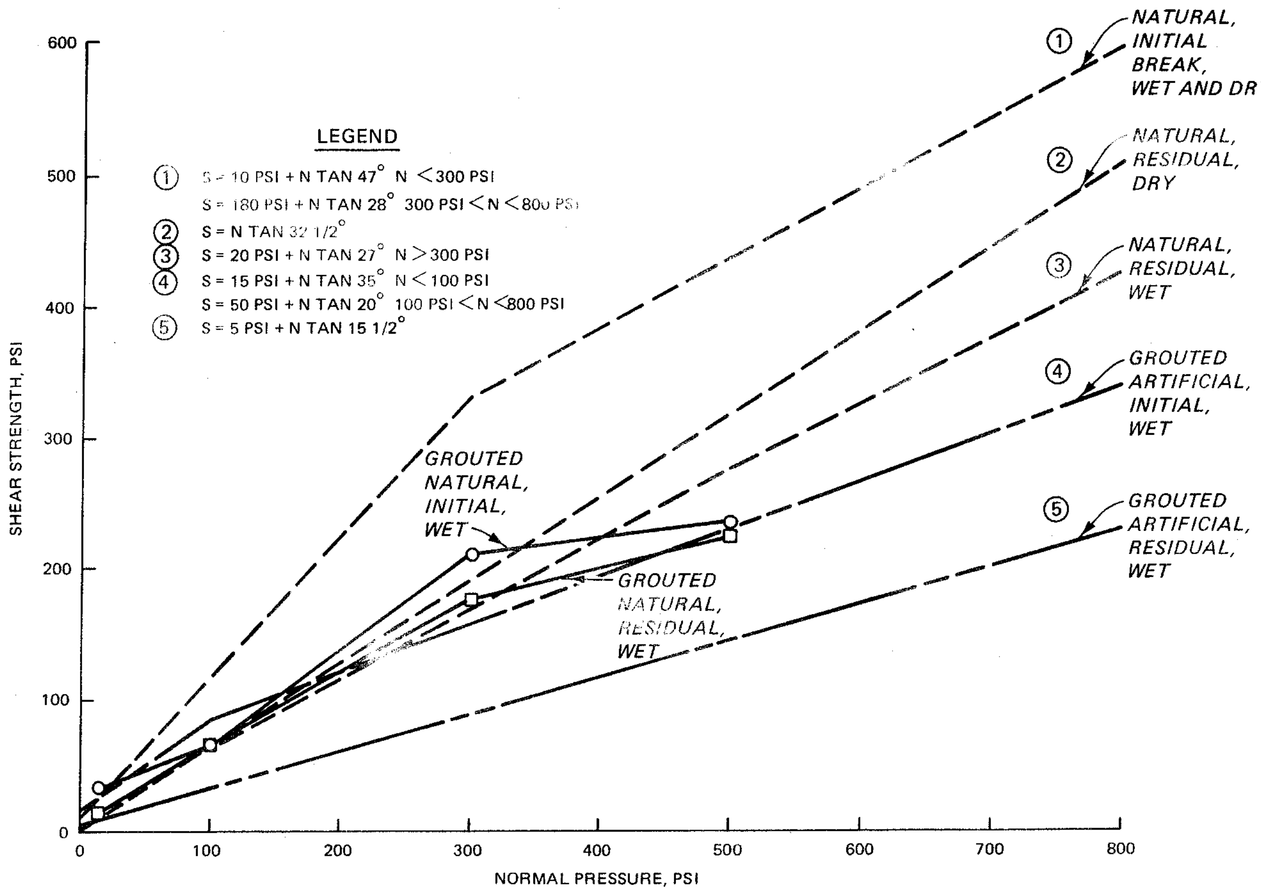


Figure 12. Comparison of shear strengths for natural joints, grouted natural joints, and grouted artificial joints (after Deere and Coulson 1971)

practice, only open joints will receive grout; therefore, direct application of the laboratory test results obtained from tighter fissures should be done with caution.

Evaluation of Grouting

74. Many methods to evaluate effectiveness of grouting have been attempted. These evaluations include drilling and sampling boreholes, excavating test pits, and driving adits for direct access to the grouted rock. Indirect methods include geophysical testing by seismic profiling, electrical resistivity, and borehole and surface radar. Plate load tests, borehole jacks, and flatjacks have also been used to assess grouting effectiveness. Perhaps the most basic method is quantitative evaluation of unit takes during split spacing, which is a combination of monitoring and evaluation. To be

most effective, an evaluation should be conducted in the field while grouting is in progress.

75. The level of monitoring and evaluation is an economic decision. As Huck and Waller (1982) discuss, the cost of monitoring and testing should be balanced with the increased design efficiency of the grouting effort. The risk of grouting failure and its consequences should determine the relative mix of design conservatism with monitoring and testing effort.

76. Monitoring and evaluation, using unit takes, have been done throughout the history of grouting. Grant (1964a, 1964b) compares unit take with hole spacing and with primary, secondary, tertiary and so forth grouting. He shows how unit take can be expected to decrease as the grout hole spacing closes in. More recently, Millet and Engelhardt (1982) put the unit take concept into a matrix formulation to evaluate grouting in progress. The method readily identified zones needing additional treatment with a minimal number of proof holes.

77. The other evaluation methods, employing core sampling, jacking tests, or geophysical testing, are typically conducted subsequent to the grouting operation. The primary disadvantage to postgrout testing would be in remobilizing the contractor, if additional treatment is required.

78. Evaluations of the increase in rock mass stiffness by application of consolidation grouting methods have been documented by many authors. Table 3 summarizes some of the documented cases. Allas and Savinskaya (1972) noted greater improvement in the more severely fractured rocks than in the rock with a higher initial soundness. In the fractured zones, elastic wave velocity increased by 100 to 150 percent; whereas, the velocity increased only by 10 to 20 percent in the higher quality rock. Grouting tended to equalize the wave velocity and resulted in an overall increase in deformation moduli. Static plate loading tests were conducted to determine moduli values while wave velocities were determined by seismic profiling methods, a dynamic test.

79. Serafim and Guerreiro (1974) presented in situ deformability test results on highly anisotropic schists. The jacking tests, performed inside galleries, were oriented at a range of inclinations with respect to the schistosity planes, before and after grouting. They found the grouting treatment accentuated the anisotropy. As shown in Figure 13, prior to grouting the greatest moduli were measured parallel to the schistosity ($\alpha = 90_0$). Post-grouting analysis showed an increase of about 100 percent in modulus at the

Table 3

Summary of Rock Mass Improvement from Grouting

Reference	Property Measured	Percent Increase from Grouting	Test Methods	Comments
Allas and Savinskaya (1972)	Deformation modulus E	27 to 143	Plate load test	
	Wave velocity V_p	2 to 146	Seismic	
Serafim and Guerreiro (1974)	Deformation modulus E	15 to 25	Flat jack	Normal to schistosity
		100	Flat jack	Parallel to schistosity
		110 to 115	Flat jack	33° to 55° from normal
Soejima and Shidomoto (1970)	Wave velocity V_p	61 to 83	Seismic	
	Deformation modulus E	23 to 100	Not stated	
Bogoslovsky and Ogilvy (1973)	Electrical resistivity	400 to 500	Resistivity	Contours of resistivity prepared showing areal extent and variation with depth. Resistivity changes with time recorded.
Neff, Sager, and Griffiths (1982)	Deformation modulus E	0	Borehole jack	Data shown are for basaltic flow
	Wave velocity V_p and V_s	106	3-D Acoustic	
	Wave velocity V_p and V_s	126	Crosshole Seismic	Breccia
Scalabrini, Carugo, and Carati (1964)	Wave velocity V_p	20 to 82	Seismic	
	*Deformation modulus E	43 to 204	Seismic	

* Seismic directly reads V_p. V_s is read or calculated. The deformation modulus E must be calculated.

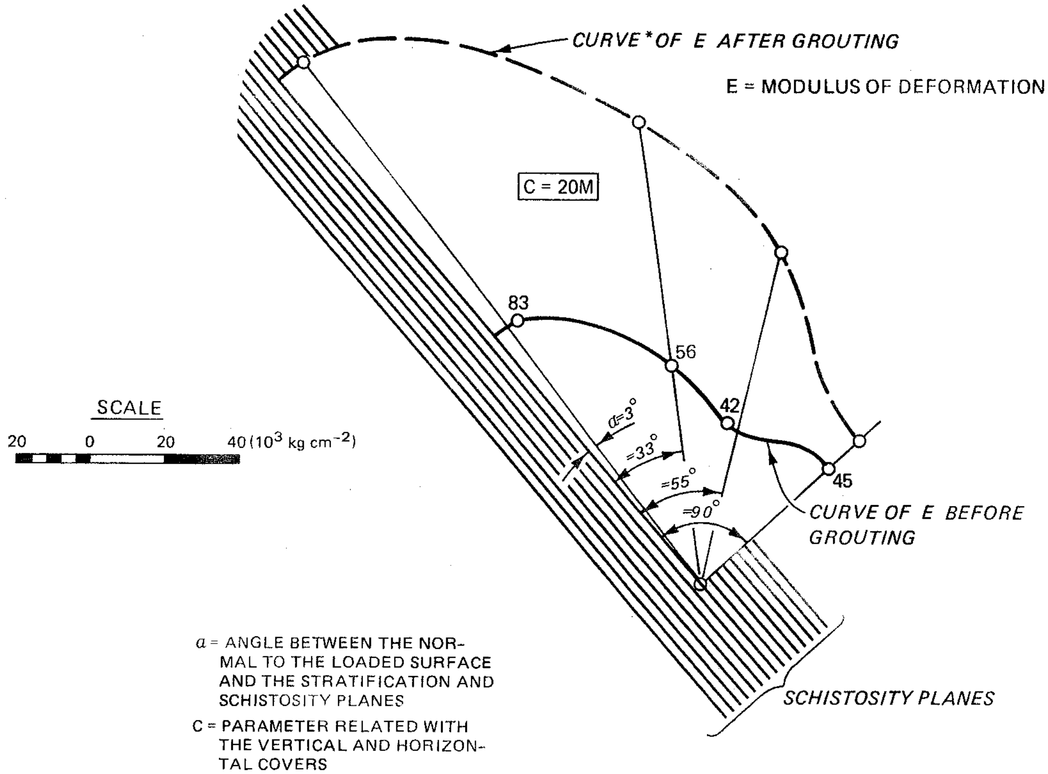


Figure 13. Anisotropy of deformability before and after grouting (after Serafim and Guerreiro 1974)

same orientation while the modulus normal to schistosity increased only 25 percent.

80. Most investigators mention a decrease in nonrecoverable deformation of grouted rock, which is caused by closure of cracks and fissures under applied load. Grouting fills these cracks; thus they are unable to close. Rock response to loading more closely approximates elastic behavior after effective grouting.

81. Grout monitoring and evaluation are important to the quality of the final product and are interrelated. An effective monitoring effort is a method of evaluating the grouting operation as it is being executed.

PART V: CASE HISTORIES

Little Goose Lock and Dam

82. Differential monolith movements during lock-full conditions led to concrete spalling and waterstop damage between monoliths at the Little Goose Lock and Dam. The observed movement resulted in part from differential foundation compression. Repair of the lock consisted of remedial consolidation grouting, waterstop joint repair, and repair of spalled monolith joints. Prior to initiating the rehabilitation program, a test grout program was undertaken for the purpose of determining the most effective procedures for future contract work. This part summarizes the remedial grouting operations.

83. The Little Goose Lock and Dam is on the Snake River, about 70.3 miles upstream from the confluence with the Columbia River, in Columbia County, Washington. The dam is near the southern border of the Columbia Plateau. The Snake River has cut a valley about 1,200 to 1,500 ft deep into the plateau uplands. A thick series of basalt flows underlie the area and are partially intercalated with sedimentary deposits, flow breccias, and other volcanic rock types. Basalt lava flows comprise the major portion of the rock mass. Flow breccias, contact planes between breccia and basalt lava, and sediments represent a small percentage of the stratigraphic section. The depositional nature of the individual flows over a hummocky surface and nonuniform movement before solidification resulted in a wide flow thickness variation between adjacent test holes and outcrops. The flows generally dip downstream (westerly) toward the Pasco Basin. Various basaltic lava flows have been correlated, identified, and numbered along the river valley. Results of these studies can be found in the various design memoranda describing bedrock geology at the project.

84. Two general fracture patterns appear to be near horizontal and near vertical in the flows. Both sets of fracture patterns are limited in areal extent but may be continuous over the relatively small areas affected by remedial grouting measures.

85. The buttress stems are founded on a relatively competent basalt flow layer underlain by a flow breccia layer. The lock wall deflections are

related to a lower load-carrying capacity of the flow breccia than was assumed in the design stages.

86. The severity of the lock wall deflections was monitored by structural instrumentation and leveling surveys. Differential movements between adjacent monoliths of 0.25 in. to 0.30 in. were recorded with each lockage. A permanent outward deformation of more than 0.75 in. with a continuing deflection independent of lockage cycles had been noted from survey data.

87. The flow breccia is generally a loose, unconsolidated material. Drilling and grouting procedures were modified because of frequent caving in portions of the grout holes through the breccia. The EX diameter grout holes were terminated when drilling fluid was lost or the hole caved, and they were grouted immediately. Although this procedure increased the number of hookups and required redrilling through a grouted zone, grout take was increased and the quality of grouting increased.

88. The foundation rock is overlain by approximately 40 ft of random rockfill. A pneumatic drill drove BX casing, fitted with a drive shoe, through the rock-fill into sound rock. A 1-1.2-in.-diam black pipe (wrought iron) was inserted into the BX casing, and the casing was withdrawn. The black pipe was cemented in place. EX diameter grout holes were then drilled with the black pipe serving as casing and, subsequently, as a grout nipple. The grout holes were drilled on 4-ft centers in a grid pattern. The foundation stabilization plan and sections are shown in Figure 14.

89. Grout holes were water tested prior to grouting. Generally, 12 ft^3 of water was pumped into the hole and time was recorded. A thin 6:1 water-cement ratio grout was injected immediately after the water test. The mix was thickened gradually until a sharp drop in the rate of solids injected was noted. Pumping was then continued with this mix until refusal was obtained. Refusal pressure and flow rate criteria were not mentioned in the references available. A final report covering the completed repairs has not yet been published.

90. The grout was mixed with Type III portland cement. Intrusion-aid fluidifier was added during latter stages of the grouting program. Water for the grout mix was taken directly from the Snake River. In general, grout take was low in the competent basalt, averaging about 0.35 bag/lin ft of grout hole during the test program. Large grout takes were noted in four grout holes in

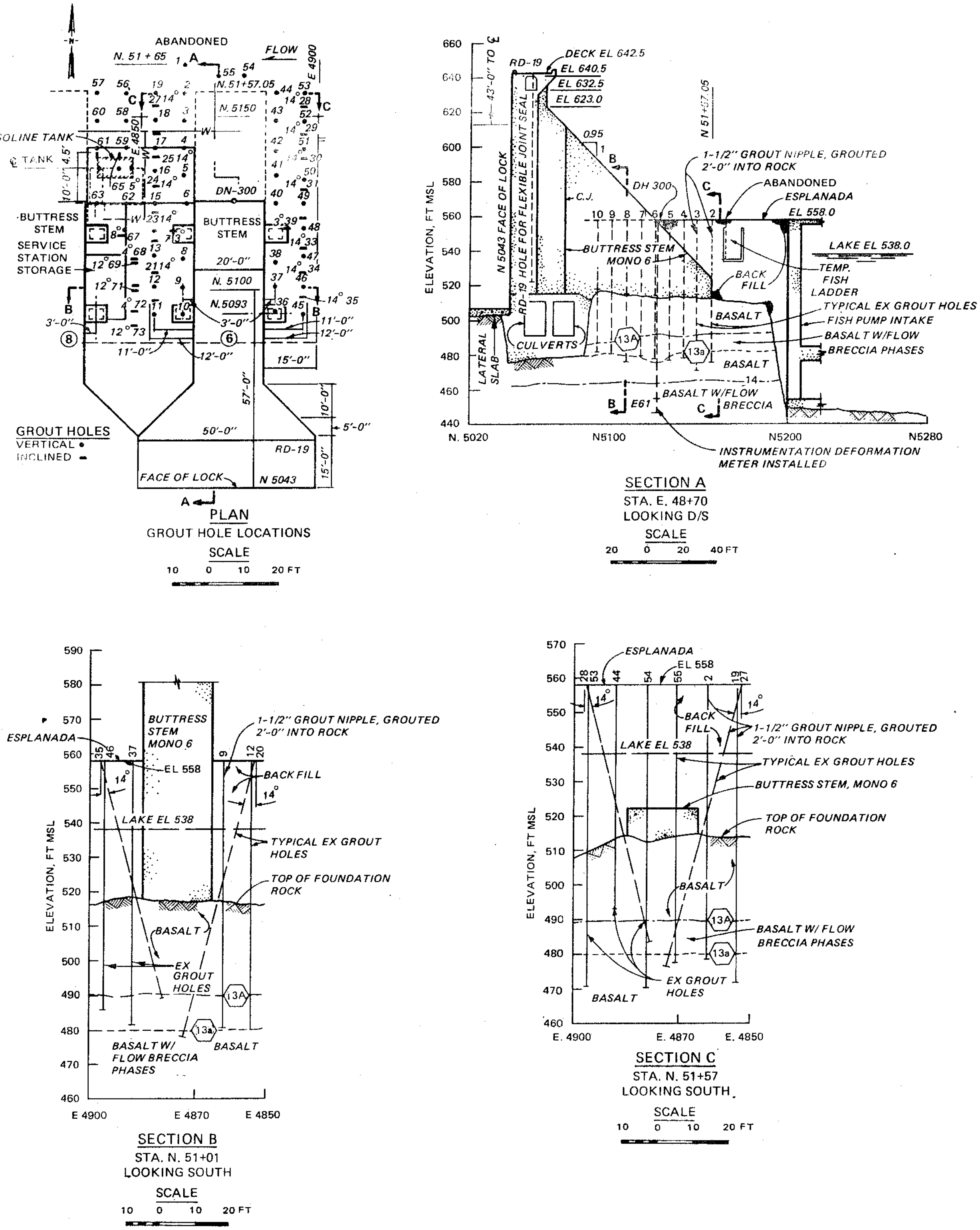


Figure 14. Stabilization plan and sections, Little Goose Lock and Dam

"competent" basalt, representing 10 percent of grouted footage. Cement placed in the four holes was 54 percent of the total cement placed.

91. The flow breccia experienced larger grout takes, as expected. On the average, 3.59 bags/lin ft of grout hole were injected into flow breccia. The use of Intrusion-aid, in recommended quantities, facilitated grout placement. The injection rate slowed substantially when the fluidifier was omitted. This was considered to be an additional method of grouting control. However, if the fluidifier were routinely omitted after a hole has experienced a large take (to reduce the injection rate), it would seem to reduce the quality of grouting by reducing the amount of cement that otherwise could have been injected. As a control method, fluidifier addition or omission should be carefully considered, and perhaps it should only be withheld when very large takes indicate grout flow into adjacent areas not requiring grout.

92. Monolith deflections at Little Goose Lake Lock and Dam have been monitored since completion of the trial grouting and foundation stabilization. Initial data indicate that the cement grouting program has reduced movement of the monoliths and has provided additional stability to the foundation. Specific conclusions from the trial grouting program were:

a. The injection of cement grout into the flow breccia in quantities sufficient to stiffen the most poorly consolidated materials is feasible. The final success of this operation is not yet known.

b. Stopping drilling at the first sign of caving or at water loss was a complete success. A statistical analysis showed a definite trend to higher grout takes in the resulting shorter holes.

c. The hole spacing appeared to be adequate and no further changes in spacing or pattern were required.

d. Use of Intrusion-aid admixture at the recommended strength appears to be beneficial. The admixture allows placing grout at a thicker mix than without admixture, and may also increase the quantity of cement that can be placed in the formation.

e. Water tests do not have much bearing on the ultimate results of grouting a hole except for the observation that where the hole will not take water it will not take grout.

93. A chemical test grouting program was also undertaken. The chemical grouting in similar geological material was performed downstream from the navigation lock. Two chemical grout compounds were injected--300 gal of TACSS T-025NF with 0.3 gal of C-855 accelerator were furnished by TJK, Inc.

and Associates; about 250 gal of AM-9, produced by American Cyanamid, was injected in the second test. TACSS T-25NF grout in core samples was in the liquid stage 20 days after testing and hardened only after it was exposed to air. No AM-9 grout was observed in the core recovered in the AM-9 test area (US Army Engineer District, Walla Walla 1978). Test grouting with chemical grouts in the given geology yielded unacceptable results and was not recommended for the proposed rehabilitation program.

Savage River Dam Spillway Rehabilitation*

94. Remedial consolidation grouting was performed as a part of the rehabilitation of the spillway weir at the Savage River Dam. This rehabilitation included a change in the design floor from 50,000 cfs to 100,000 cfs. Additional work included installation of poststressed tendons to tie the spillway to the rock, thereby permitting the design flow to occur safely. In the model study, the first three monoliths failed by overturning.

95. The dam is on the Savage River, about 4.5 miles upstream of the confluence with the North Branch of the Potomac River, in Garrett County, Maryland. Savage River Dam is an earth and rockfill dam with a concrete side-channel spillway weir. Other pertinent features of the dam are presented in Table 4. Rehabilitation work was conducted from 20 October 1971 through 29 January 1972.

96. The Savage River Dam is between Backbone and Big Savage Mountains on a typical, young, poorly developed floodplain in the Plateau region of the Appalachians. The dam and spillway are founded on the Loyalhanna Member of the Greenbrier Formation (Mississippian Age). The bedrock is a medium- to fine-grained limestone with dispersed quartz sand. The formation dips to the southeast into the left abutment at 10 to 15 deg. Major joint orientations are near vertical and strike to the northeast and northwest, roughly parallel and perpendicular to the spillway.

97. Consolidation grouting beneath the spillway weir was the first step in the rehabilitation work. Grouting was done along three lines, which for this discussion will be designated A, B, and C.

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* This discussion includes information received in a personal communication with W. Trautwein, 1984.

Table 4

Savage River Dam, Pertinent Data*

Drainage Area	
Savage River (above Dam)	76 sq miles
Crabtree Creek	29 sq miles
Total	105 sq miles
Reservoir	
Acreage (at spillway-crest el **)	360 acres
Storage (at spillway-crest el)	20,000 acre-ft
Dam	
Length	1,050.0 ft
Height above streambed	184.0 ft
Crest el	1,497.5
Type of construction	Earth and rockfill
Spillway	
Length of crest	320.0 ft
Crest el	1,468.5
Discharge capacity (24.3 ft over crest)	97,200 cfs
Type of spillway	Side-channel
Outlet Structure	
Tunnel	
Type	Horseshoe-shaped
Diameter	10 ft
Length	1,170 ft
Discharge capacity (reservoir water surface at spillway crest)	4,850 cfs
Length of conduit transition:	
Upstream	9.25 ft
Downstream	16.75 ft
Side Gates	
Type	Hydraulically operated
Number	2 twin sets
Size	4 by 10 ft

* After "Report on Rehabilitation of Spillway Weir" (US Army Engineer District, Baltimore 1973).

** In this report, elevations are in ft NGVD.

Grout holes were drilled to a minimum depth of 10 ft beneath the spillway slab.

98. Grout holes were spaced 20 ft on center along the three lines. Adjacent lines were spaced at 7 ft, with holes staggered to allow about 10 ft of spacing to holes in adjacent lines. Grout holes on lines A and C were rotary drilled with an NX diamond bit and core barrel through the concrete spillway. The holes were then advanced by percussion drilling to final depth with a 3-in. down-hole air hammer. Grout holes in line B were cored the entire depth. The last seven holes in line C were cored full depth because of air equipment breakdowns.

99. Drilling fluid (air or water) loss was observed in 15 of the 48 grout holes drilled. In nine of these holes, drilling fluid was lost in drain tiles encountered while drilling line C. One hole in line A and four holes in line B experienced drilling fluid loss in foundation rock. Drilling fluid was lost in a 0.2-ft-thick sand layer at the concrete-rock interface in one hole in line B. Each hole was pressure tested prior to grouting. Water was observed flowing from the rock-concrete interface between sta 3+00 and sta 3+10 during drilling and pressure testing of hole B-10.

100. Grouting was initiated after drilling and pressure testings were completed for the entire line. As a construction expedient, however, holes B-14, B-15, and B-16 were grouted immediately after these holes were drilled. Two depth zones were grouted in lines A and B; whereas, only one zone was grouted in line C because of the drain tile encountered during drilling of line C. Zone I extended from the hole bottom upward to a depth of 5 ft into rock. Zone II overlaid Zone I and extended upward into the concrete.

101. The mixing plant was a 16-ft^3 concrete mixer. The mixer emptied into a holding tank, equipped with an agitating mechanism to promote mix uniformity and hold the mix in suspension. The grout pump was a screwfeed type. The grouting pressure was regulated by pump speed and monitored at the grout header. The grout header was a "T," with a recirculating line leading back to the holding tank. The constant grout circulation provided by this arrangement prevented line clogging and allowed accurate pressure control.

102. Type II portland cement was mixed with reservoir water initially in a 3:1 water-cement ratio (by volume). The water-cement ratio was varied in response to encountered conditions. The thickest mix used was 0.6:1 water-cement ratio for backfilling the grout holes. The refusal criteria were

1 ft^3 of grout in 5 min at maximum allowable gage pressures of 10 psi in Zone I and 4 psi in Zone II. Pressures were not corrected to absolute or adjusted for height of grout column, ground-water conditions, or any other factors. Experience showed maximum grout take occurs when the maximum pressure was applied immediately and maintained throughout the grouting period.

103. Chemical dye was mixed with the grout injected in lines B and C. Since lines A and C were grouted before line B was drilled, one check of grouting effectiveness included observation of grout from A and C lines obtained in recovered core samples in B line drilling. The drill logs for line B show grout in joints from both A and C line grouting operations, as distinguished by the presence or absence of dye.

104. A total of 789, 716, and 619 lin ft of grout hole was drilled for lines A, B, and C, respectively. Total grout placed, including backfill for lines A, B, and C, was 325, 785, and 694 ft^3, respectively.

105. Holes in line A were drilled into an area grouted during spillway construction. The grout take was less in line A than on the other lines indicating the area to be tight. However, original grouting apparently missed high-angle (near-vertical) joints, as grout was observed rising to the surface where joints daylighted at the rear of the spillway weir when A and B lines were grouted.

106. The foundation was considered to be effectively grouted to a depth of 10 ft below the spillway floor over 90 percent of the area; however, this conclusion was supported only by observations during grouting. One such observation was of the areas of high take during line C grouting that accepted little grout during line B grouting. This led to the conclusion that areas or voids missed by one grout line were intersected by another line. The analysis or evaluation appears to be based on qualitative observations. Structural performance appears to be adequate, although the spillway has not been subjected to design flood.

John Day Lock and Dam

107. Concrete spalling at the John Day Lock and Dam was observed in two lock chamber monoliths during routine dewatering and inspection in 1975. Extensive cracking near the middle of the lock accompanied the observed

spalling. By 1979, cyclic filling and emptying operations of the lock caused the cracking to extend into six monoliths. One of the probable reasons for crack propagation was the overstressing of an under-reinforced culvert section as a result of foundation deformation.

108. Repair consisted of consolidation grouting of the foundation rock strata and installation of epoxy-grouted posttensioned anchors. The first repair phase, foundation consolidation grouting, will be presented below.

109. The John Day Lock and Dam is on the Columbia River about 215 miles from the river mouth and about 110 miles east of Portland, Oregon, within the North Pacific Division, Portland District (NPP). The lock is north of the dam spillway on the right, or the Washington shore. The lock consists of individual concrete monoliths designed as gravity structures.

110. In general, the foundation rock units are composed of very hard, fine-grained, dark gray, dense basalt. A dense columnar basalt is the immediate founding stratum and overlies a flow breccia. A characteristic pattern of prominent joints, formed during the cooling of the molten basalt lava into rock, gives the appearance of "columns" of rock where the basalt is exposed. Many of these vertical joints are open and transmit ground water. Additional sets of fractures formed during and after rock formation result in an overall fractured and blocky appearance. Vesicles, or small unconnected void spaces resulting from the rock-forming process, are scattered throughout the basalt. Some vesicles are filled with secondary minerals.

111. The flow breccia rock layer is composed of solidified lava and ejected volcanic materials in the form of ash, lapilli (volcanic fragments), and bombs, often all welded together. The flow breccia comprises the original contact surface between separate lava flows. The overriding lava flow cooled and solidified into a weak crust when chilled rapidly by the previously deposited basalt. Rapid cooling, weathering, alteration, and the heterogeneous nature of the rock have produced a brownish-colored rock layer that is soft, weak, crumbly, highly fractured (or brecciated), slightly cemented, and rough textured. Because of the nature of its deposition, both the top and bottom flow breccia contacts with solid rock are very irregular. Thickness of the flow breccia unit ranges from 25 to 40 ft. The flow breccia rock also contains a large number of void spaces, many of which are interconnected. The physical nature of the flow breccia is analagous to a mass of dirty, crushed rock.

112. Basalt below the flow breccia unit is very similar to the upper basalt but exhibits less columnar structure. The filling and emptying culvert sections are founded on the upper surface of this lower basalt layer.

113. Numerous laboratory tests were performed on small-diameter core samples to assess the various rock properties. The flow breccia exhibited lower shear strength, unit weight, and elastic parameters than did the basalts. Special emphasis during design and construction was placed on evaluating the flow breccia's load-carrying capacity and ability to serve adequately as a foundation material. The flow breccia rock was considered suitable for founding the lock's floor slab. The culvert sections were founded on basalt. In Design Memorandum No. 16 for the lock, recommendations were made to relax the original proposal requiring a minimum of 25 ft of basalt be maintained between gravity section foundations and the upper basalt/flow breccia contact. The structural "as built" drawings and the foundation report indicate the basalt thickness (breccia to foundation) ranges from 8 to 36 ft. Monoliths 11, 19, 21, and 23 on the riverside apparently have less than 25 ft of basalt cover over the breccia, and on the landside, monoliths 12, 14, 16, and 20 also have less than 25 ft of basalt overlying the flow breccia.

114. Based on postconstruction instrumentation, very little vertical deformation occurred in the foundation during lock filling and emptying, so distribution of vertical stress within the foundation was probably not a problem. There is a considerable degree of anisotropy in the rock mass deformation characteristics, normal in layered rocks, and accentuated by weathering effects and the natural discontinuities present in the remaining rock mass between the lock and the spillway. Lateral restraint of the foundation was minimized by the excavated configuration, and stress relief fractures undoubtedly occurred as a result of blasting and removal of the surrounding rock. The rock mass deformation modulus was probably decreased laterally by the prolonged exposure period of the foundation rock during construction before concrete placement. This combination of factors and the presence of the less rigid flow breccia allowed considerable lateral, elastic deformation to occur within the rock mass between the riverside monoliths and the river. This elastic response, in return, allowed the riverside monoliths to deflect during lock filling. The monoliths returned to their original position upon lock

emptying because of the elastic response of the rock and the reorientation of the monolith gravity load resultant.

115. The flow breccia is a weathered, low density material exhibiting considerable variation in hardness, shear strength, and deformation parameters. Laboratory testing showed a wide range in strength and elastic modulus values, mainly from the lateral and vertical petrologic and mineralogic variations within the flow breccia. Portions of the breccia have been altered to palagonite, a soft, brownish or greenish-black alteration product of basaltic glass composed primarily of the clay mineral montmorillonite. The strength and consolidation properties of palagonite are more similar to a soil than to a rock. Based on past explorations and mapping, the palagonite zones are infrequent and small in lateral extent. Recovery of small-diameter core samples is very difficult in palagonite zones, and some bias against these very soft zones has occurred during the foundation investigations. At the other extreme from the palagonite zones, the flow breccia is characterized as a highly vesicular basalt and exhibits much higher strength, hardness, etc. The overall strength and deformation properties of the entire flow breccia rock mass are also controlled to a large extent by the discontinuities within the layer. The flow breccia, as a mass, also exhibits very high transmissibility to water because of the interconnection of a large percentage of void spaces within the rock. Permeability varies according to the degree of void space interconnection and the presence of fines within the voids. Faults and shears within the flow breccia decrease the permeability of the rock within their zones of influence.

116. During a lock outage in March 1979, the flow rate through the flow breccia was estimated at about 1,700 gpm, based on computation of the discharge rate from the sump pump used to maintain both filling and emptying culverts in the dewatered condition. Except for an estimated 10 percent from leakage of pool water through the upstream gate, the major amount of water discharged from the sump pump during the lock outage was assumed to have flowed through the breccia. Using 1,700 gpm as a rate of flow Q through the breccia layer, a permeability estimate on the order of 4×10^{-3} ft/min was made.

117. Grouting in flow breccia has been successful in the past, as discussed previously in this report. Grout travel is predictably erratic, however because of the variations in permeability.

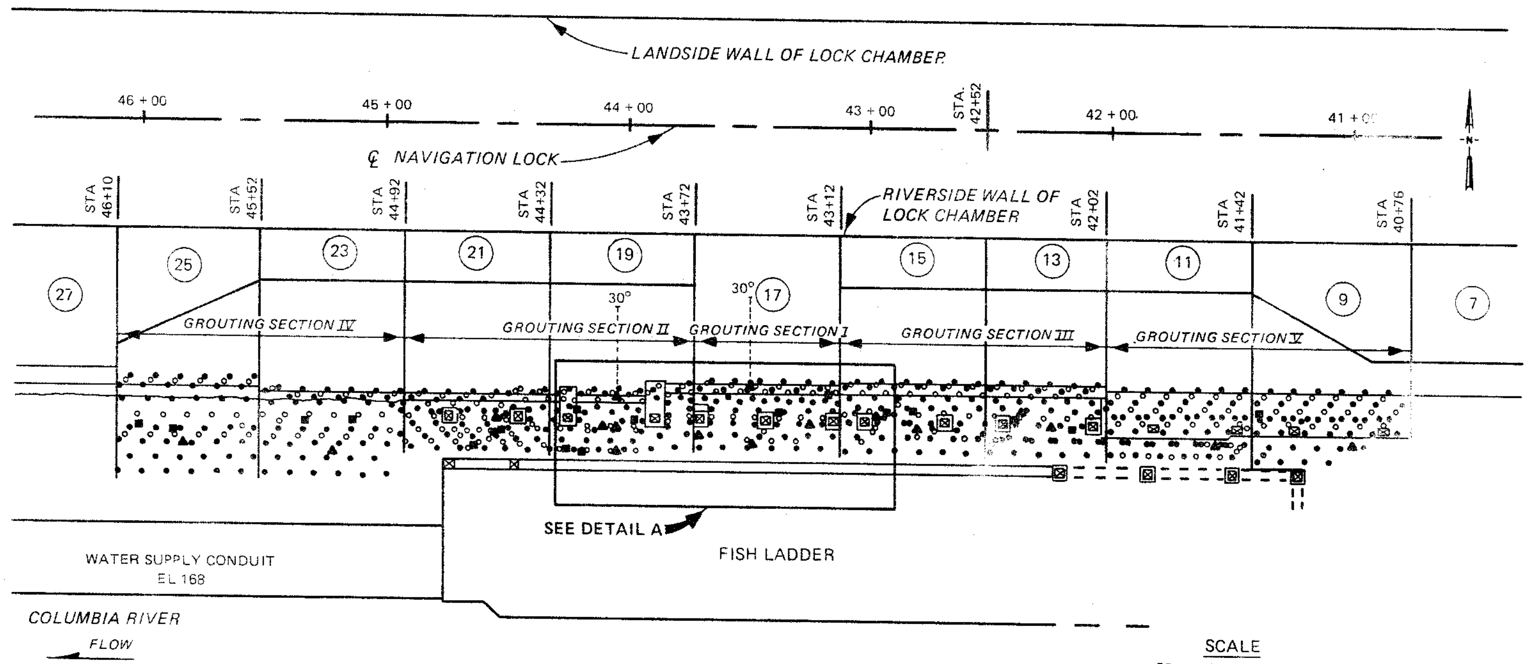
118. Primary holes were arranged in essentially six staggered lines paralleling the lock wall. Because of space limitations, grout hole lines were close together on the surface, and the holes were drilled at angles ranging from 5 to 25 deg from vertical to provide coverage of the entire foundation rock mass (see Figure 15). The holes in each primary line were generally on 10-ft centers, while spacing between lines was typically 6 ft. Some secondary grout holes were planned between lines of primary holes to assure complete coverage in the rock closest to the lock. A number of relatively low-angle holes were included to intercept the high number of near-vertical joints in the basalt. Fishladder support pillars and other obstacles required altering the grout hole pattern at some locations.

119. In general, the grout holes were planned to extend 10 ft below the contact between the flow breccia layer and underlying basalt or to the elevation of the base of the culvert portion of the structure (el 115), whichever was deeper. The typical hole depth was approximately 75 ft. The program of planned grout holes required a maximum grout travel between holes of 10 to 12 ft in the direction perpendicular to the lock to achieve complete coverage at the lowest elevation. The option of split spacing was provided in the contract to reduce the required grout travel if it was determined during the work that the planned pattern was not adequate.

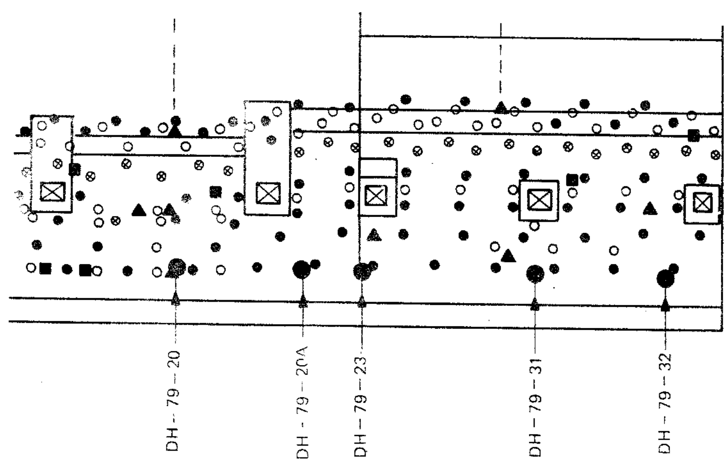
120. The rock mass was divided into two zones for purposes of the grouting program. The first zone extended from the top of the hole to the upper basalt-flow breccia contact, approximately 30 ft. The second zone extended through the flow breccia layer and into the underlying basalt to the bottom of the hole. Grouting Zone 1 in a single stage was permitted. Grout stages were limited to 5-ft intervals in the flow breccia layer. Previous grouting experience in this type of material had indicated a need for short stages in the flow breccia to achieve good penetration and adequate control. A thin grout mix with a water-cement ratio of 5:1 (5 parts water to 1 part cement, by volume) was specified for the start of grout injection to penetrate the fine openings expected in the rock.

121. Grout takes for the lock repair foundation grouting were expected to average about 0.4 sacks of cement per linear foot of hole in Zone 1 (basalt) and 1.2 sacks per linear foot in Zone 2 (primarily flow breccia).

122. Grout holes were EW size, 1-1/2 in. in diameter, and were drilled with rotary drilling equipment. As planned, the first holes drilled and



PLAN OF GROUT HOLE LOCATIONS



DETAIL A, LOCATION OF GROUT SURVEY HOLES

LEGEND

- (19) MONOLITH NUMBER
- ☒ FISH LADDER COLUMN
- ● PRIMARY GROUT HOLE
- ○ SECONDARY GROUT HOLE
- ⊗ LOW-ANGLE PRIMARY GROUT HOLE DIRECTED TOWARD RIVER
- ■ TERTIARY GROUT HOLE
- ▲ EXPLORATORY HOLE DRILLED DURING GROUTING CONTRACT
- ⬤ GROUT SURVEY HOLE DRILLED PRIOR TO GROUTING

NOTES: GROUT HOLES ARE EW 11.5 IN. IN DIAMETER.

GROUT HOLE PATTERN VARIED LOCALLY FROM DETAIL DRAWINGS SHOWN (SEE PLAN). GROUT HOLE COLLARS WERE OFFSET IF NECESSARY TO AVOID DAMAGING EXISTING INSTRUMENT HOLES

SPACINGS SHOWN IN DETAIL DRAWINGS ARE HORIZONTAL DISTANCES

HORIZONTAL SURVEY CONTROL WAS BASED ON STATIONING, AS GIVEN FOR EACH MONOLITH JOINT

SEE SECTIONS ON PLATES 6 AND 7 FOR INCLINATIONS OF GROUT HOLES.

LOGS OF EXPLORATORY HOLES ARE SHOWN IN PLATES 8 THROUGH 10

Figure 15. Grout and survey hole locations

grouted were the row of primary holes furthest from the lock. These holes were drilled and grouted to the full depth opposite each monolith to form a grout barrier, which permitted more efficient grouting in the holes closer to the lock. Grout pressures in these barrier holes were kept low to minimize undesired grout travel. The remaining lines of primary holes, progressing toward the lock, were then drilled and grouted in Zone 1 and this procedure was repeated for Zone 2. The secondary holes were then drilled and grouted in Zones 1 and 2. In general, drilling in Zone 2 did not begin in an area until the holes had been completely grouted in Zone 1.

123. Prior to performing injection grouting in each stage, the hole was washed and a water pressure test performed. Grout injection was normally started with a batch of grout having a water-cement ratio of 5:1. The mix was then thickened cautiously at the discretion of the construction inspector. It was discovered during performance of the work, however, that difficulties in grout injection were encountered with mixes thicker than about 3:1; thus, thick mixes were seldom used. Type I portland cement was used in the grout, and a fluidifier was added to facilitate penetration into the finest fractures possible. Grout pressures were kept low, sometimes as low as 10 psi at the top of the hole, especially in Zone 1, to avoid damaging nearby fishladder structures and to prevent surface venting. All grout injection was performed with the water level in the lock at tailwater elevation to avoid grouting against high seepage pressures in the rock when the lock was full and to avoid grouting while the rock was in a state of maximum strain. The refusal criterion for each grout stage was 2 ft^3 of cement per hour, determined over a 10-min period at the refusal pressure determined by the depth of that stage. All holes were grouted individually.

124. Additional secondary holes were assigned, where the rock appeared more open, based on grout takes in the primary and assigned secondary holes. Locations for these additional holes were determined by the split-spacing method. A few tertiary grout holes were drilled to determine the overall completeness of the grouting job. Tertiary hole locations were selected at random, and grout takes were compared with those for the primary and secondary grout holes to determine if a satisfactory reduction in open voids had been achieved. Throughout the contract period, exploratory core borings were drilled to help determine the effectiveness of the grouting program in filling rock voids.

125. This tabulation shows a simple comparison of the key variables in the contract:

Variable	Predicted	Actual
Total drilling footage		
Zone 1	23,000	24,438
Zone 2	17,000	19,054
Total grout take (sacks)		
Zone 1	9,200	3,540
Zone 2	20,400	29,640
No. of drill holes		
Primary	402	397
Secondary	166	213
Tertiary	27	22

126. Based on a review of all the past and recent foundation investigations performed at the navigation lock site, "most reasonable" values have been selected for the significant engineering properties exhibited by the different rock types. These values are shown in Table 5.

Table 5

John Day Navigation Lock, Remedial Repair Summary of Significant Bedrock Engineering Properties

Property	"Most Reasonable" Value: Flow Breccia	"Most Reasonable" Value: Basalt
Field unit weight	137 lb/ft^3	180 lb/ft^3
Dry density	121 lb/ft^3	--
Natural water content	14%	--
Unconfined compressive strength	1,600 psi	20,000 psi
Triaxial shear strength	C = 240 psi, Ø = 22°	--
Modulus of deformation	0.5×10^6 psi (72×10^6 psf)	7×10^6 psi ($1,000 \times 10^6$ psf)
Poisson's ratio	0.17	0.25
Permeability	10×10^{-4} ft/min	2×10^{-4} ft/min

127. The US Army Engineer Waterways Experiment Station (WES), Vicksburg, Mississippi, performed a series of tests to determine the

deformation modulus of in situ materials at John Day Lock and Dam before and after grout injection.

128. The in situ deformation moduli were determined from measurements obtained with the Goodman borehole jack, three-dimensional (3-D) acoustic velocity logger, and downhole and crosshole seismic techniques. The tests were conducted in holes drilled for the survey by the NPP. Location of each survey hole is shown in Figure 15. The geotechnical survey provided quantitative descriptions of the pregrout and postgrout in situ deformation moduli of the breccia zone to complement laboratory data obtained by the NPP laboratory.

129. The geophysical tests were conducted in five borings that were located along an 82-ft-long line parallel to the lock wall and fishladder and about 129 ft riverward of the lock center line, as shown in Figure 15. The borings were made to nominal depths of 80 ft by a CP-60 casing-mounted drilling unit using NW diamond bits (2,88-in.-hole diameter) and water as the drilling fluid. The station locations were:

Boring	Station
DH 79-22	43+21
DH 79-21	43+43
DH 79-23	43+72
DH 79-20A	43+82
DH 79-20	44+03

Prior to conducting the survey, borings DH 79-20 and DH 79-20A caved and could not be used for the survey.

130. Six Goodman borehole jack tests, numbered GJ-1, 2, 3, 4A, 4B, and 5, were performed in boring DH 79-22. Four tests, numbered GJ-6, 7A, 7B, and 8, were performed in boring DH 79-21. A typical plot of hydraulic pressure versus displacement is shown in Figure 16. Moduli were calculated by two methods--the "Hustrulid" method and the "Goodman" method, giving upper and lower bounds to moduli values, respectively. Values of test elevation, calculated and assumed Poisson's ratios, contact angles, pressure-displacement slopes, and lower bound and upper bound moduli are presented in Table 6. Calculated moduli are plotted to depth in Figures 17 and 18. The modulus for intact basalt, by one test, falls between 1,1800,000 and 4,660,000 psi for the two methods. The modulus range for breccia, by nine tests, falls between

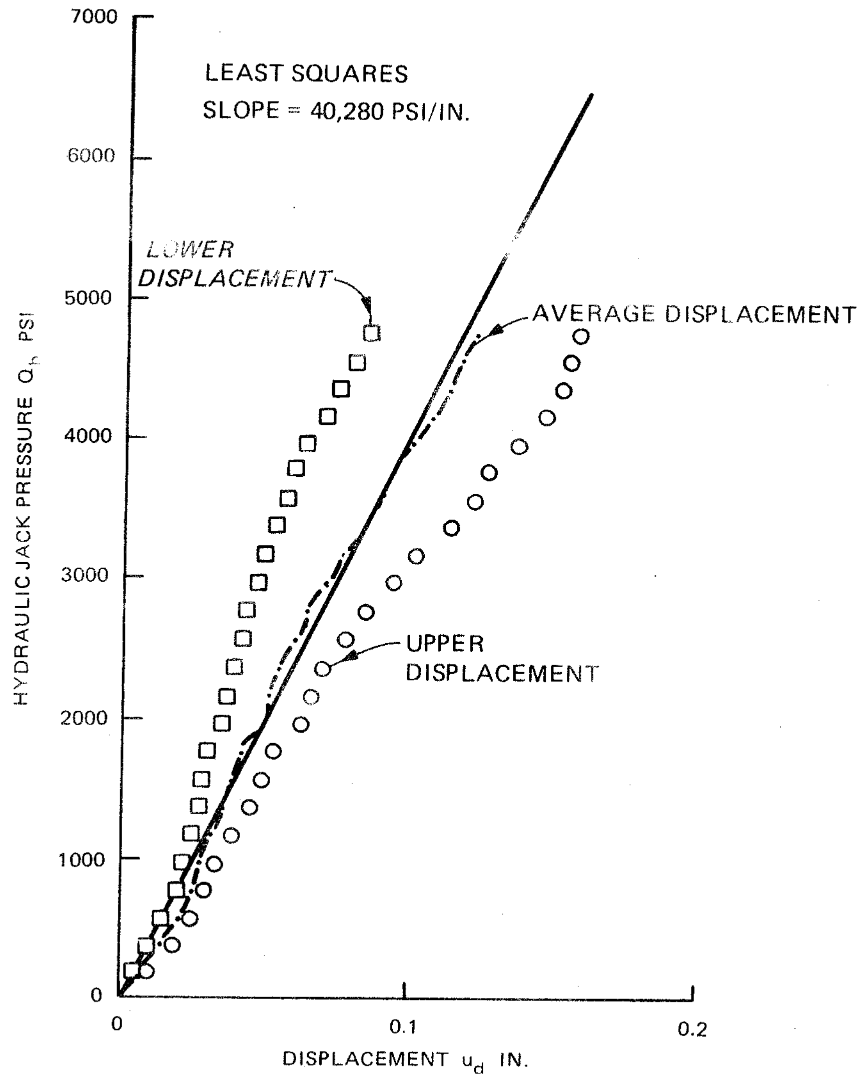


Figure 16. Typical plot of Goodman borehole jack test data pregrout survey

34,000 psi and 801,000 psi. The average lower bound modulus (Goodman method) is 85,500 psi, and the average upper bound modulus (Hustrulid method) is 339,000 psi for in situ breccia. The ratio of the lower bound moduli of breccia to basalt is 0.071. The ratio of the upper bound moduli of breccia to basalt is 0.072. For both bounds, therefore, the ratio is approximately 1:14, breccia to basalt, which is in agreement with the ratio assumed for design, 500,000:7,000,000 psi. This ratio was initially determined from the results of static tests in the laboratory. Since the Goodman jack test is the only in situ test conducted that provided data comparable to static laboratory results, this comparison is most appropriate for comparison of laboratory and in situ derived moduli. The data are comparable despite the deviation in

Table 6

Goodman Jack Test Results, Pregrout Survey

Test: Hole: el, ft: Parameter	GJ-1 DH 79-22 123.7-124.4	GJ-2 DH 79-22 133.7-134.4	GJ-3 DH 79-22 143.7-144.4	GJ-4A DH 79-22 148.7-149.4	GJ-4B DH 79-22 149.5-150.2	GJ-5 DH 79-22 159.7-160.4	GJ-6 DH 79-21 128.6-129.3	GJ-7A DH 79-21 133.6-134.3	GJ-7B DH 79-21 135.1-136.8	GJ-8 DH 79-21 141.6-142.3
Rock type	Breccia	Breccia	Breccia	Breccia	Breccia	Breccia	Breccia	Breccia	Breccia	Breccia
Poisson's ratio 3-D	0.4*	0.4*	0.4*	0.4*	0.4*	0.457	0.416	0.4*	0.4*	0.392
Contact Angle __-deg	3.5	3.5	4.8	3.5	5.2	3.5	3.5	3.5	3.5	3.5
Ratio of change in hydraulic pressure, psi to change in borehole diameter, in.	40,280	69,625	28,949	26,857	43,650	436,735	17,688	17,512	12,159	15,238
"Hustrulid" E , psi/in.	513,000	801,000	290,000	309,000	422,000	4,660,000	199,000	201,000	140,000	177,000
"Goodman" E , psi/in.	119,000	196,000	81,000	75,000	123,000	1,180,000	49,000	49,000	34,000	43,000

* Assumed value for ν , other values based on 3-D log results.

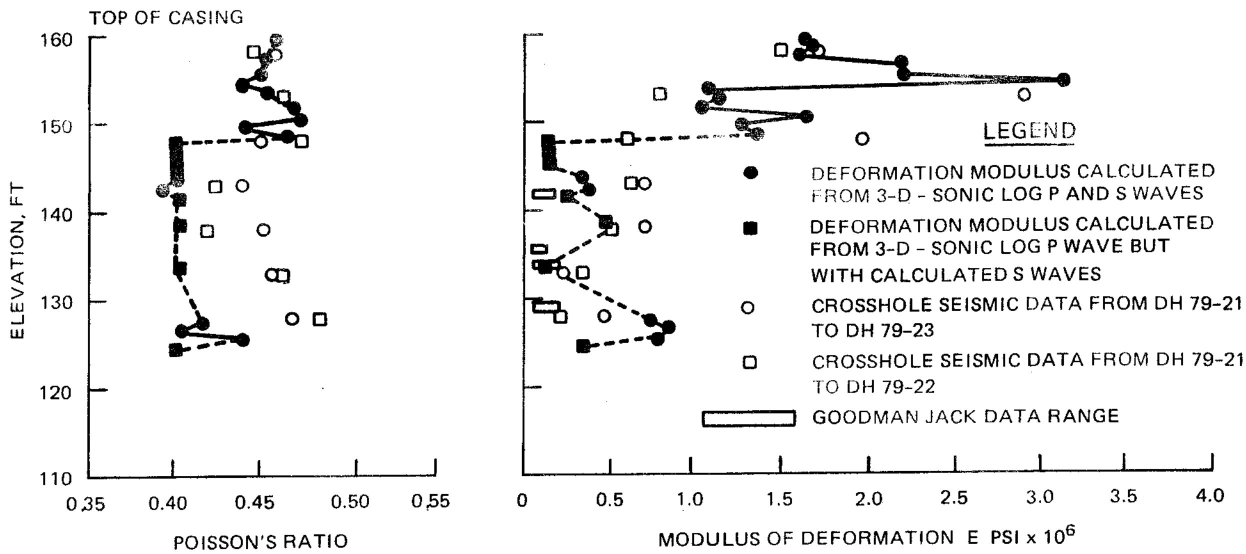


Figure 17. Elastic parameter measurements, DH 79-21, pregrout survey

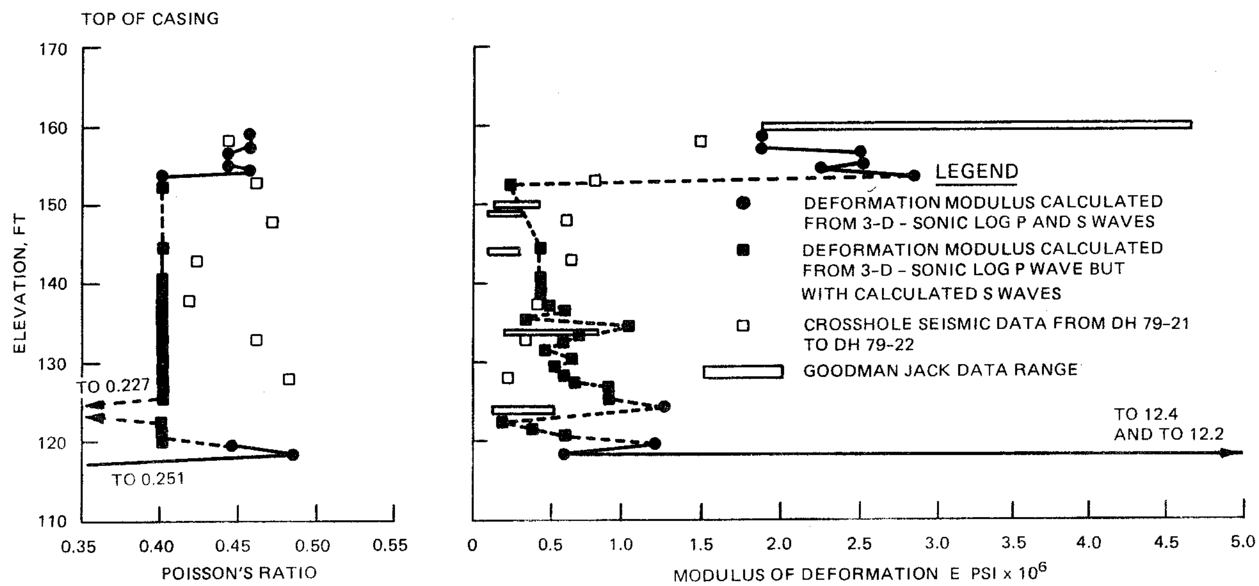


Figure 18. Elastic parameter measurements, DH 79-22, pregrout survey

magnitudes. That deviation may be representative of the difference in boundary conditions between laboratory and in situ tested volumes of rock.

131. For interpretation of the 3-D velocity log, the borehole diameter at each chosen location in the borehole is required; such data were obtained from the caliper log of each boring. The velocity of the P-wave in the borehole fluid of 5,100 ft/sec was obtained directly from the 3-D log by observing the deformation wave arrivals. The measured velocities and calculated parameters (including porosity, bulk density, Poisson's ratio, and dynamic Young's modulus from the 3-D logs) are given in Table 7. The elastic parameters are plotted to depth in Figures 17 and 18.

132. No return signal was recorded in substantial lengths of the boring. Those zones are entirely within the breccia and an unknown combination of low velocities, large numbers of attenuating fractures, and an irregular hole configuration prevented detectable signals from being received. The rock in these zones is not conducive to 3-D logging in its natural state. Any special treatment, such as grouting or casing the boring to obtain a return signal, would alter the natural rock conditions and make any possible logs totally unrepresentative of the pregrout character.

133. Density data were obtained in the laboratory on 19 samples from various elevations and locations within the breccia zone. If the voids in the samples were assumed to be 100 percent saturated with water, the average bulk density of the 19 laboratory specimens would be 139.6 lb/ft^3 with a range from 132.3 to 147.2 lb/ft^3; other samples were available but were too fragmentary for bulk density determination. The bulk density obtained by digitizing the 3-D velocity logs at 1-ft intervals average 115.9 lb/ft^3. The dynamic in situ deformation moduli were calculated using the bulk density obtained by interpreting the 3-D velocity data because the data were directly comparable in depth and location to the zones tested in the survey. The dynamic deformation moduli of the breccia calculated from 3-D data average 367,078 psi and range from 121,000 to 822,000 psi for 13 reported calculations in boring DH 79-21. Comparable results from boring DH 79-22 produce an average of 565,000 psi with a range from 181,000 psi for 1,250,000 psi in 22 reported calculations. The average for breccia in both holes is 492,000 psi and is to be compared with the Goodman jack test average results ranging between 85,500 and 339,000 psi. In the basalt, the 3-D derived moduli average 2,710,000 psi and range from 571,000 to 12,400,000 psi in 22 reported calculations.

Table 7

3-D Sonic Log Results, Pregrout Survey

DH 79-21							DH 79-22						
Elevation ft	P-Wave Velocity V_p ft/sec	S-Wave Velocity V_s ft/sec	Poisson's Ratio ν	Porosity d%	Bulk Density lb/ft^3	E Dynamic Young's Modulus psi	Elevation ft	P-Wave Velocity V_p ft/sec	S-Wave Velocity V_s ft/sec	Poisson's Ratio ν	Porosity d%	Bulk Density lb/ft^3	E Dynamic Young's Modulus psi
159.6	14,286	4,054	0.456	15.7	154.0	1,599,490	159.7	--	--	--	--	--	--
158.6	14,286	4,110	0.455	15.7	154.0	1,642,800	158.7	15,294	4,317	0.457	12.6	157.4	1,854,666
157.6	13,636	4,054	0.452	17.9	151.6	1,570,343	7.7	15,294	4,317	0.457	12.6	157.4	1,854,666
156.6	15,000	4,688	0.446	13.5	156.4	2,157,167	6.7	15,789	5,000	0.444	11.2	158.9	2,489,606
155.6	15,000	4,688	0.446	13.5	156.4	2,157,237	155.7	15,789	5,000	0.444	11.2	158.9	2,489,606
4.6	16,667	5,556	0.437	9.0	161.3	3,105,190	4.7	15,667	4,688	0.457	9.0	161.3	2,241,515
3.6	11,538	3,448	0.451	26.8	141.9	1,062,787	3.7	13,636	5,556	0.400	17.9	151.6	2,843,890
2.6	12,500	3,448	0.459	22.4	146.7	1,062,787	2.7	7,164	1,915	0.4	62.2	103.5	230,643
1.6	13,043	3,307	0.466	20.2	149.1	1,104,630	1.7	--	--	--	--	--	--
150.6	16,668	3,969	0.470	9.0	161.3	1,621,181	150.7	--	--	--	--	--	--
9.6	11,359	3,773	0.438	27.8	140.9	1,251,527	9.7	--	--	--	--	--	--
8.6	14,258	3,683	0.464	15.8	153.9	1,326,649	8.7	--	--	--	--	--	--
7.6	5,993	1,602	0.4	80.5	83.6	130,370	7.7	--	--	--	--	--	--
6.6	5,990	1,601	0.4	80.5	83.6	130,207	6.7	--	--	--	--	--	--
145.6	6,010	1,606	0.4	80.1	84.0	131,649	145.7	--	--	--	--	--	--
4.6	6,119	1,635	0.4	78.2	86.1	139,857	4.7	8,907*	2,380	0.4	44.0	123.3	424,387
3.6	6,120	2,488	0.401	78.2	86.1	324,128	3.7	--	--	--	--	--	--
2.6	6,138	2,588	0.392	77.8	86.5	350,212	2.7	--	--	--	--	--	--
1.6	7,757	2,073	0.4	55.1	111.2	290,368	1.7	--	--	--	--	--	--
140.6	--	--	--	--	--	--	140.7	8,880	2,373	0.4	44.2	123.0	420,866
9.6	--	--	--	--	--	--	9.7	8,907	2,380	0.4	44.0	123.3	424,367
8.6	9,266	2,476	0.4	41.1	126.4	470,862	8.7	8,854	2,366	0.4	44.4	122.8	417,709
7.6	--	--	--	--	--	--	7.7	9,127	2,439	0.4	42.2	125.2	452,557
6.6	--	--	--	--	--	--	6.6	10,106	2,701	0.4	35.1	132.9	589,140
135.6	--	--	--	--	--	--	135.7	8,166	2,182	0.4	50.8	115.9	385,304
4.6	--	--	--	--	--	--	4.7	12,500	3,341	0.4	22.4	146.7	995,013
3.6	5,867	1,568	0.4	82.9	81.0	121,010	3.7	10,604	2,834	0.4	32.0	136.3	665,163
2.6	--	--	--	--	--	--	2.7	9,926	2,653	0.4	36.3	131.6	561,826
1.6	--	--	--	--	--	--	1.7	9,161	2,448	0.4	41.9	125.5	456,996
130.6	--	--	--	--	--	--	130.7	10,236	2,736	0.4	34.2	133.9	609,058
9.6	--	--	--	--	--	--	9.7	9,794	2,618	0.4	37.2	130.6	5__,911
8.6	--	--	--	--	--	--	8.7	10,000	2,673	0.4	35.8	132.2	57_,951
7.6	8,398	3,181	0.416	48.5	118.4	736,150	7.7	10,498	2,806	0.4	32.6	135.6	
6.6	8,422	3,374	0.404	48.3	118.6	822,677	6.7	11,723	3,133	0.4	25.9	142.9	
125.6	9,478	3,123	0.439	39.4	128.3	781,291	125.7	11,772	3,133	0.4	25.7	143.1	
4.6	8,237	2,201	0.4	50.1	116.6	343,229	4.7	7,735	4,596	0.227	55.3	111.0	
3.6	--	--	--	--	--	--	3.7	6,537	4,596	0.011	71.2	937.0	
2.6	--	--	--	--	--	--	2.7	6,676	1,784	0.4	69.1	960.0	151,207
1.6	--	--	--	--	--	--	1.7	8,400	2,245	0.4	48.5	118.4	362,602
120.6	--	--	--	--	--	--	120.7	10,071	2,692	0.4	35.3	132.7	564,341
							9.7	11,538	3,671	0.444	26.8	141.9	1,198,892
							8.7	14,286	2,397	0.486	15.7	154.0	570,651
							7.7	21,429	8,922	0.395	0.1	171.0	1,23_,670
							6.7	20,000	11,538	0.251	2.3	168.6	12,186,906
							115.7	17,647	14,804	2.104	6.8	163.7	--
							4.7	14,803	12,610	0.823	14.1	155.7	--
							3.7	15,166	13,979	2.324	13.0	156.9	--
							112.7	16,383	14,806	1.725	9.7	160.5	--

* Where Poisson's ratio could not be determined from test results, a value of 0.4 was assumed.
** Test data could not be obtained in the zones shown above without data.

134. In a crosshole seismic test, the transit time of an acoustic wave is measured as it travels from one boring through the intervening material to a second boring. The velocity of waves in the material can be calculated from the transit time and the distance. Crosshole seismic measurements were obtained with explosive blasting caps suspended in the transmitter borehole. Three-axis oriented geophones were suspended and wedged by steel bow-springs against the receiver borehole walls. The blasting caps were detonated at 5-ft-depth intervals starting at el 8 in boring DH 79-21. The receivers were placed at the same elevations in borings DH 79-23 and DH 79-22. The borings were assumed to be vertical. Between borings DH 79-21 and DH 79-22, the horizontal distance was assumed to be 22 ft. The spacing between DH 79-21 and DH 79-23 was assumed to be 29 ft. From single shots, wave records were received that included both the initial arriving P-wave and the later arriving S-wave. Velocities calculated from the transit times were compared with results obtained from the 3-D velocity logs and correlated to possible geologic cross sections to verify the interpretive assumption of straightline horizontal ray paths. The transit times and calculated wave velocities assuming nonrefracted ray paths in the crosshole seismic test compare favorably with results interpreted from the 3-D logs. Based on cross sections, refracted ray paths were considered to be inconsequential to the calculated results.

135. The results from the crosshole seismic tests are given in Table 8 and shown graphically versus depth in Figures 17 and 18. The solid basalt produced an average dynamic deformation modulus of 1,980,000 psi in a range from 1,472,000 to 2,834,000 psi obtained in four crosshole measurements. The breccia produced an average dynamic deformation modulus of 504,600 psi in a range from 201,500 to 759,600 psi obtained in 10 crosshole measurements. These values were obtained using bulk densities calculated from wave velocities rather than from laboratory data. The calculated densities from crosshole seismic data for basalt average 156.0 lb/ft^3 and for breccia average 121.6 lb/ft^3. The latter is to be compared with average 3-D log results of 115.9 lb/ft^3. Note that the 3-D log tests a region within inches of the borehole wall; whereas, the crosshole seismic tests measure an effective average of material between borings.

Table 8

Crosshole Seismic Results, Pregrout Survey

	DH 79-21 to DH 79-23						DH 79-21 to DH 79-22					
Elevation ft	P-Wave Velocity V_p ft/sec	S-Wave Velocity V_s ft/sec	Poisson's Ratio ν	Porosity n, percent	Bulk Density lb/ft^3	E Dynamic Young's Modulus psi	P-Wave Velocity V_p ft/sec	S-Wave Velocity V_s ft/sec	Poisson's Ratio ν	Porosity n, percent	Bulk Density lb/ft^3	E Dynamic Young's Modulus psi
158.0	14,500	4,142	0.456	0.15	154.8	1,680,000	12,571	4,000	0.444	0.22	147.2	1,470,000
153.0	19,333	5,167*	0.462	0.04	167.3	2,830,000	11,000	2,940*	0.462	0.30	138.5	760,000
148.0	14,500	4,461	0.448	0.15	154.8	1,940,000	11,000	2,588	0.471	0.30	136.5	592,000
143.0	8,923	2,974	0.438	0.44	123.4	681,000	8,000	2,933	0.422	0.52	114.6	608,000
138.0	9,666	2,900	0.451	0.38	129.7	687,000	7,333	2,750	0.418	0.60	105.9	493,000
133.0	6,823	1,901	0.458	0.67	98.4	225,000	8,000	2,146	0.461	0.52	114.6	335,000
128.0	9,666	2,367	0.468	0.38	129.7	463,000	8,800	1,600	0.483	0.45	122.3	202,000

* V_s derived from ν calculated from 3-D data.

136. The average moduli determined by the pregrout survey are given in this tabulation.

Test Method	Modulus, psi	
	Solid Basalt	Breccia
Goodman, jack (static, localized)	1,180,000-4,660,000	85,500-339,000
3-D sonic log (dynamic, localized)	2,710,000	492,000
Crosshole seismic (dynamic, large volume)	1,980,000	504,600

The deformation moduli determined from the Goodman jack, the 3-D velocity measurements, and the crosshole seismic measurements are plotted versus depth in Figures 17 and 18.

137. The postgrout, in situ moduli were determined from measurements obtained with the Goodman borehole jack, the 3-D acoustic velocity logger, and crosshole seismic techniques similar to the pregrout survey. The postgrout survey was performed to duplicate the successful phases of the earlier testing program with the addition of Ludgeon (water injection) tests. The borings used in the pregrout survey were filled with sand for preservation prior to the grouting operations and then cleaned out by washing with water for reuse in the postgrout survey. Tests were conducted in these borings: DH 79-22, DH 79-21, DH 79-23, and DH 79-20A. DH 79-20A required two washings and subsequent checking with the caliper logger before access below a depth of 52 ft was made possible. The first caliper log of DH 79-23 indicated a restriction at a depth of 37 ft that required clearing by a drill-rig crew. As testing progressed, a blockage resulted in DH 79-21 that prevented using the 3-D logger or crosshole seismic equipment below a depth of 55 ft. The Goodman borehole jack data can be presented graphically as plots of hydraulic pressure versus displacement; a typical plot is shown in Figure 19. The results of Goodman jack tests are shown in Table 9 and Figures 20-23. At several test locations, the displacements indicated excessively irregular borehole wall conditions, and the jack was depressurized and raised or lowered a few inches repeatedly until a satisfactory borehole configuration was shown by the indicated platen displacements. The results of the jack tests are in the form of ranges of deformation modulus--the lower number representing the Goodman analysis used for softer rock, and the upper number representing the Hustrulid

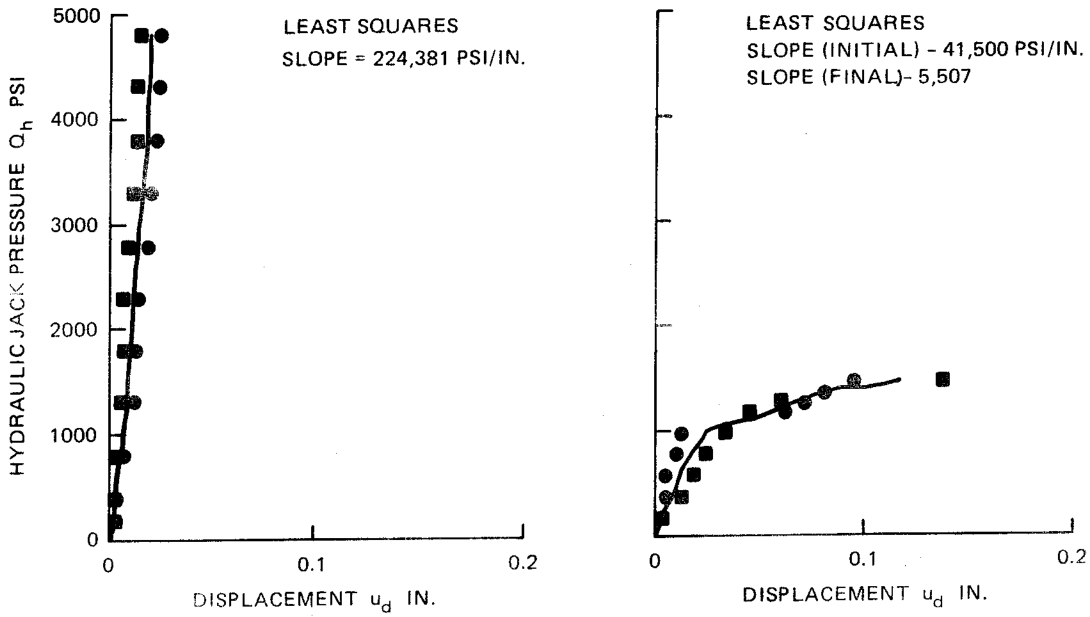


Figure 19. Typical plots of Goodman borehole jack test results, postgrout survey

analysis for hard rock. Incorporating all jack tests, the modulus range for intact basalt and the grouted breccia are as follows:

Material	Modulus Range, psi
Intact basalt (6 measurements)	691,900-2,320,800
Breccia (16 measurements)	85,800-329,400

138. The measured sonic velocities and calculated parameters including bulk density, Poisson's ratio, and Young's, shear, and bulk moduli are shown in Table 10 from the 3-D log data. Comparison moduli from pregrout and postgrout surveys calculated from the 3-D sonic log data are shown in Figures 24 and 25. Results of the 3-D sonic log test are also shown in Figures 20-23.

139. Qualitative examination of the postgrout 3-D records shows a substantial improvement in the received acoustic signals allowing more complete data retrieval than possible prior to grouting. Some zones, however, still fail to allow passage of usable acoustic signals, particularly for the 6-ft-transmitter-to-receiver spacing. Where only the P-wave are detectable, a value of 0.4 was assumed for the dynamic Poisson's ratio. The same assumption was used for the pregrout data reduction.

Table 9

Goodman Borehole Jack Test Results, Postgrout Survey

Elevation ft	Material	Poisson's Ratio ν	Change in Hydraulic Pressure, psi / Change in Hole Diameter, in.	Lower Bound Modulus psi	Upper Bound Modulus psi
			DH 79-20A		
151.8-152.4	Basalt	0.24	224,381	657,500	2,251,800
129.3-129.9	Breccia	0.4*	41,501	116,500	477,500
129.3-129.9	Breccia	0.4*	5,507	15,500	63,400
125.8-126.4	Breccia	0.14	9,171	28,200	128,600
125.5-126.1	Breccia	0.14	2,493	7,700	35,000
125.3-125.9	Breccia	0.41	15,974	44,500	181,500
119.3-119.9	Breccia	0.42	29,560	81,900	299,400
116.3-116.9	Basalt	0.19	63,009	192,500	773,500
Average	Basalt			425,000	1,512,650
Average	Breccia			49,050	197,570
			DH 79-23		
146.5-147.1	Breccia	0.43	79,560	219,000	880,600
138.5-139.1	Breccia	0.4*	13,369	37,500	153,800
138.0-138.6	Breccia	0.4*	29,723	83,500	308,700
133.0-133.6	Breccia	0.4*	22,622	63,500	260,300
126.0-126.6	Breccia	0.4*	88,572	248,700	823,600
117.5-118.1	Basalt	0.47	220,694	589,500	1,950,000
108.5-109.1	Basalt	0.43	114,592	315,400	1,146,200
Average	Basalt			452,450	1,548,100
Average	Breccia			130,440	309,280
			DH 79-21		
165.3-165.9	Basalt	0.4*	304,329	854,600	2,830,000
			DH 79-22		
159.4-160.0	Basalt	0.45	568,052	1,541,900	4,973,300
139.4-140.0	Breccia	0.4*	65,448	183,800	679,500
134.4-135.0	Breccia	0.42	45,554	126,200	461,400
126.4-127.0	Breccia	0.17	6,092	18,700	84,300
125.9-126.5	Breccia	0.21	27,094	82,500	367,200
125.4-126.0	Breccia	0.15	4,725	14,500	66,000
Average	Breccia			85,140	331,680

* Assumed value for ν , others based on 3-D log results.

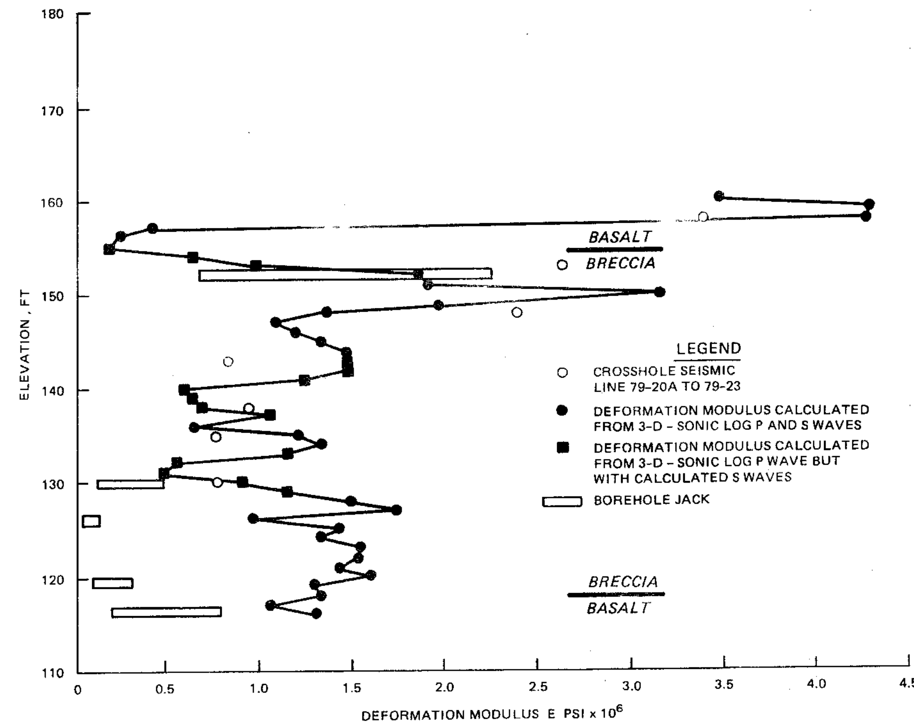


Figure 20. Postgrout deformation measurements, DH 79-20A

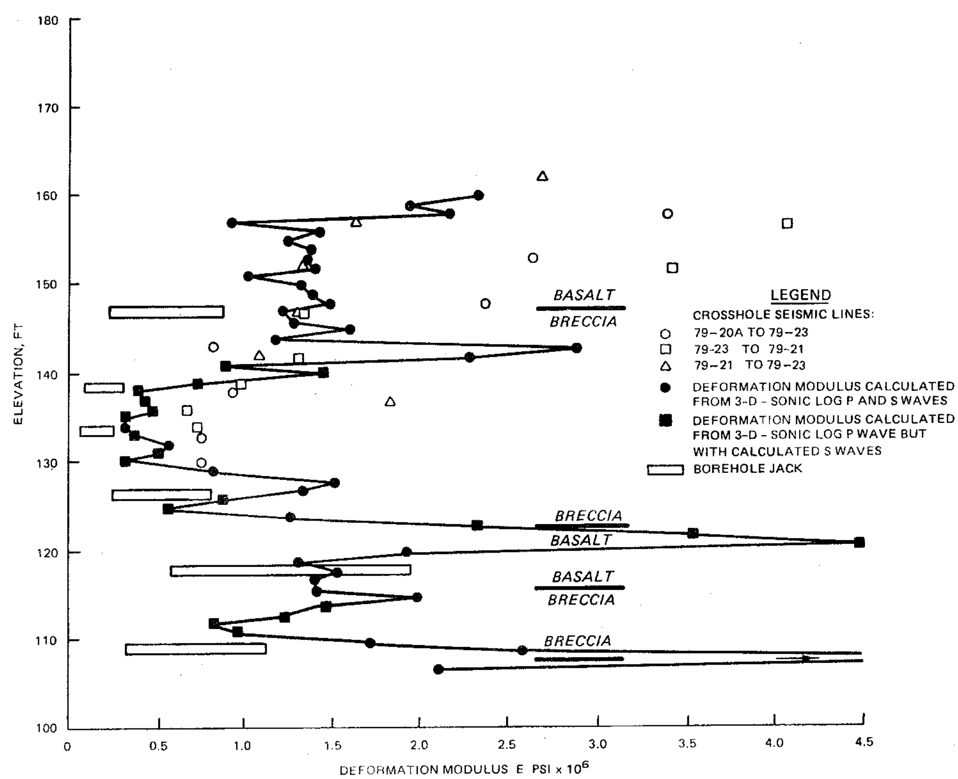


Figure 21. Postgrout deformation measurements, DH 79-23

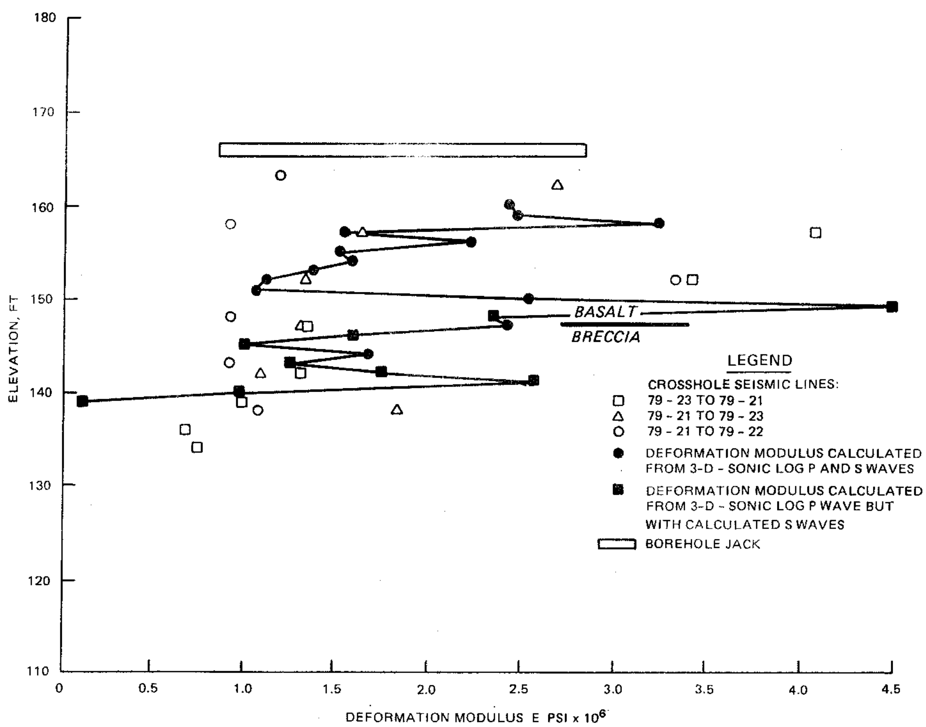


Figure 22. Postgrout deformation measurements, DH 79-21

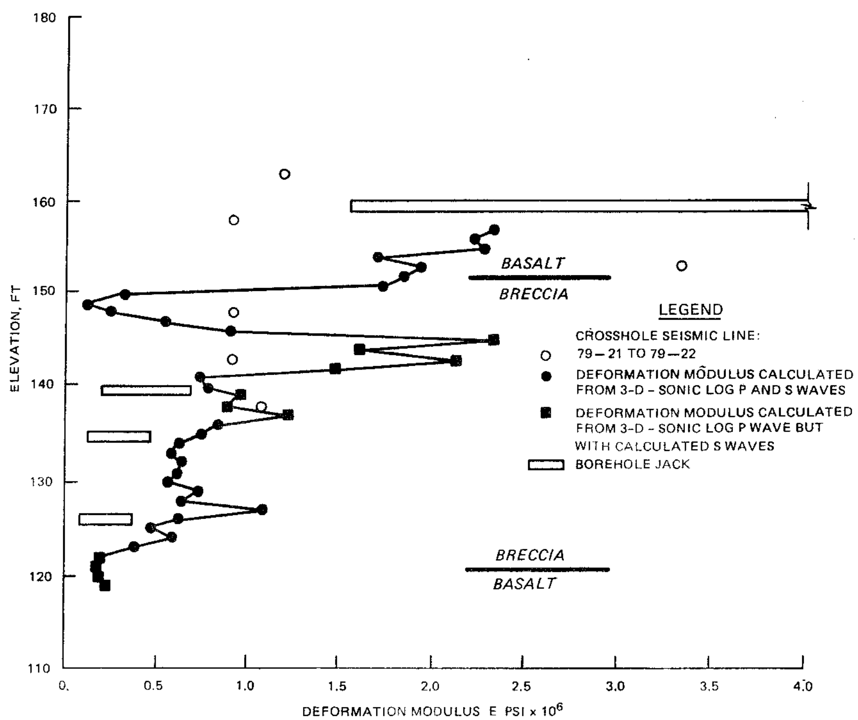


Figure 23. Postgrout deformation measurements, DH 79-22

Table 10

3-D Sonic Log Results, Postgrout Survey

Elevation ft	P-Wave Velocity V_p ft/sec	S-Wave Velocity V_s ft/sec	Poisson's Ratio ν	Bulk Density lb/ft^3	Young's Modulus E psi	Shear Modulus G psi	Bulk Modulus K psi
			DH 79-20A				
160	11,737	6,648	0.264	143.6	3,460,000	1,369,000	2,441,000
159	13,952	6,973	0.334	153.2	4,285,000	[illegible]	4,290,000
158	14,940	6,797	0.369	156.6	4,274,000	1,560,000	5,458,000
157	5,357	3,371	0.172	71.8	413,000	176,000	210,000
156	4,918	3,954	--	60.0	237,000	202,000	43,000
155	5,085	2,076	0.4	64.8	169,000	60,000	281,000
154	7,500	3,062	0.4	109.6	621,000	222,000	1,034,000
153	8,824	3,602	0.4	123.7	969,000	346,000	1,616,000
152	9,677	4,984	0.319	130.8	1,849,000	701,000	1,707,000
151	9,961	4,990	0.332	132.9	1,902,000	714,000	1,892,000
150	17,647	5,546	0.445	163.8	3,141,000	1,087,000	9,552,000
149	17,647	4,348	0.468	163.8	1,960,000	668,000	10,111,000
148	10,345	4,054	0.409	135.5	1,354,000	480,000	2,487,000
147	9,677	3,659	0.417	130.8	1,070,000	378,000	2,138,000
146	8,108	4,225	0.314	116.6	1,179,000	449,000	1,055,000
145	10,345	4,000	0.412	135.5	1,320,000	468,000	2,504,000
144	10,345	4,225	0.400	135.5	1,458,000	521,000	2,428,000
143	10,345	4,223	0.4	135.5	1,459,000	521,000	2,433,000
142	10,345	4,223	0.4	135.5	1,459,000	521,000	2,433,000
141	9,677	3,951	0.4	130.8	1,233,000	440,000	2,054,000
140	7,317	2,987	0.4	107.2	578,000	206,000	963,000
139	7,542	3,079	0.4	110.1	630,000	225,000	1,051,000
138	7,692	3,140	0.4	111.9	666,000	238,000	1,111,000
137	9,091	3,711	0.4	126.1	1,049,000	375,000	1,748,000
136	5,957	4,363	--	85.1	644,000	349,000	185,000
135	7,422	4,661	0.174	108.6	1,195,000	509,000	612,000
134	7,806	4,738	0.208	113.3	1,326,000	549,000	758,000
133	9,375	3,827	0.4	128.4	1,136,000	406,000	1,893,000
132	7,143	2,916	0.4	104.9	539,000	192,000	898,000
131	6,818	2,783	0.4	100.1	468,000	167,000	781,000
130	8,571	3,499	0.4	121.4	898,000	321,000	1,469,000
129	9,375	3,827	0.4	128.4	1,136,000	406,000	1,897,000
128	9,091	4,544	0.333	126.1	1,498,000	562,000	1,499,000
127	12,000	4,416	0.422	144.9	1,733,000	609,000	3,687,000
126	6,818	4,414	0.139	100.1	958,000	421,000	443,000
125	10,714	4,110	0.414	137.9	1,420,000	502,000	2,744,000
124	10,345	4,000	0.412	135.5	1,320,000	468,000	2,504,000
123	11,111	4,225	0.415	140.2	1,528,000	540,000	3,013,000
122	11,538	4,167	0.425	142.6	1,522,000	534,000	3,382,000
121	11,111	4,054	0.423	140.2	1,414,000	497,000	3,070,000
120	12,500	4,167	0.437	147.3	1,586,000	552,000	4,228,000
119	10,345	3,947	0.415	135.5	1,288,000	455,000	2,520,000
118	11,538	3,846	0.437	142.6	1,308,000	455,000	3,488,000
117	7,500	4,167	0.277	109.6	1,048,000	410,000	782,000
118	7,500	5,000	0.100	109.6	1,300,000	591,000	542,000
			DH 79-22				
157	15,559	4,831	0.447	158.4	2,307,000	797,000	7,207,000
156	14,831	4,762	0.443	156.2	2,204,000	764,000	6,391,000
155	14,745	4,826	0.440	155.9	2,255,000	783,000	6,266,000
154	13,636	4,225	0.447	152.0	1,693,000	585,000	5,315,000
153	16,667	4,348	0.463	161.5	1,927,000	658,000	8,797,000
152	15,789	4,286	0.460	159.1	1,841,000	630,000	7,713,000
151	12,500	4,348	0.431	147.3	1,719,000	601,000	4,163,000
150	4,918	3,529	--	60.0	312,000	161,000	98,000
149	4,633	3,907	--	53.0	127,000	174,000	19,000
148	4,918	3,931	--	60.0	246,000	200,000	46,000
147	5,660	3,880	--	78.9	541,000	256,000	204,000
146	7,317	3,846	0.309	107.2	895,000	342,000	782,000
145	12,500	5,103	0.4	147.3	2,316,000	827,000	3,861,000

(Continued)

(Sheet 1 of 3)

Table 10 (Continued)

Elevation ft	P-Wave Velocity V_p ft/sec	S-Wave Velocity V_s ft/sec	Poisson's Ratio ν	Bulk Density lb/ft^3	Young's Modulus E psi	Shear Modulus G psi	Bulk Modulus K psi
			DH 79-22 (Continued)				
144	10,714	4,374	0.4	137.9	1,593,000	569,000	2,655,000
143	12,000	4,899	0.4	145.0	2,101,000	751,000	3,502,000
142	10,345	4,223	0.4	135.5	1,459,000	521,000	2,433,000
141	9,375	3,062	0.440	128.4	748,000	260,000	2,088,000
140	10,000	3,045	0.449	133.2	772,000	266,000	2,517,000
139	8,824	3,602	0.4	123.7	969,000	346,000	1,616,000
138	8,571	3,499	0.4	121.4	898,000	321,000	1,496,000
137	9,677	3,951	0.4	130.8	1,233,000	440,000	2,044,000
136	8,824	3,318	0.418	123.7	833,000	294,000	1,686,000
135	7,500	3,400	0.371	109.6	749,000	273,000	965,000
134	6,522	3,352	0.321	95.4	611,000	231,000	567,000
133	6,250	3,393	0.291	90.7	581,000	225,000	464,000
132	6,522	3,399	0.314	95.4	624,000	238,000	558,000
131	6,977	3,148	0.372	102.5	601,000	219,000	784,000
130	6,122	3,381	0.281	88.3	558,000	218,000	423,000
129	7,143	3,477	0.345	104.9	736,000	274,000	790,000
128	6,522	3,419	0.311	95.4	630,000	241,000	554,000
127	9,580	3,699	0.412	130.1	1,084,000	384,000	2,063,000
126	6,250	3,499	0.272	90.7	609,000	239,000	445,000
125	5,455	3,528	0.140	74.2	454,000	199,000	211,000
124	6,000	3,643	0.208	86.0	595,000	246,000	340,000
123	5,172	3,471	--	67.1	380,000	174,000	155,000
122	5,263	2,149	0.4	69.5	194,000	69,000	323,000
121	5,085	2,075	0.4	64.8	169,000	60,000	281,000
120	5,172	2,111	0.4	67.1	181,000	64,000	301,000
119	5,357	2,187	0.4	71.8	207,000	74,000	346,000
			DH 79-21				
160	11,538	5,382	0.362	142.8	2,431,000	892,000	2,942,000
159	11,501	5,441	0.356	142.4	2,465,000	909,000	2,850,000
158	14,143	5,907	0.394	153.9	3,230,000	1,158,000	5,095,000
157	25,000	3,704	0.489	175.6	1,548,000	520,000	22,976,000
156	23,077	4,478	0.480	173.2	2,218,000	749,000	18,894,000
155	17,647	3,797	0.476	163.9	1,504,000	510,000	10,328,000
154	17,647	3,896	0.474	163.8	1,581,000	536,000	10,286,000
153	16,667	3,659	0.475	161.5	1,373,000	466,000	9,054,000
152	13,636	3,409	0.467	152.0	1,117,000	381,000	5,587,000
151	11,538	3,448	0.451	142.6	1,061,000	366,000	3,607,000
150	15,000	5,124	0.434	156.7	2,545,000	887,000	6,421,000
149	16,667	6,804	0.4	161.5	4,515,000	1,612,000	7,525,000
148	12,500	5,103	0.4	147.3	2,316,000	827,000	3,861,000
147	13,636	5,098	0.419	152.0	2,417,000	852,000	4,959,000
146	10,714	4,374	0.4	137.9	1,593,000	569,000	2,655,000
145	8,824	3,602	0.4	123.7	969,000	346,000	1,616,000
144	8,333	5,343	0.151	119.0	1,686,000	733,000	805,000
143	9,677	3,951	0.4	130.8	1,233,000	440,000	2,054,000
142	11,111	4,536	0.4	140.2	1,742,000	622,000	2,903,000
141	13,043	5,325	0.4	149.7	2,563,000	915,000	4,272,000
140	8,824	3,602	0.4	123.7	969,000	346,000	1,616,000
139	4,702	1,902	0.4	53.4	119,000	42,000	198,000
			DH 79-23				
160	12,819	5,067	0.407	148.7	2,318,000	823,000	4,172,000
159	10,524	4,903	0.361	136.7	1,930,000	709,000	2,320,000
158	12,823	4,904	0.414	148.7	2,182,000	771,000	4,245,000
157	8,824	3,488	0.407	123.7	914,000	325,000	1,644,000
156	21,429	3,614	0.485	170.9	1,430,000	481,000	16,283,000
155	21,429	3,371	0.487	170.9	1,246,000	419,000	16,367,000
154	20,000	3,571	0.484	168.5	1,375,000	463,000	13,918,000
153	18,750	3,571	0.481	166.2	1,354,000	457,000	11,992,000

(Continued)

(Sheet 2 of 3)

Table 10 (Concluded)

Elevation ft	P-Wave Velocity V_p ft/sec	S-Wave Velocity V_s ft/sec	Poisson's Ratio ν	Bulk Density lb/ft^3	Young's Modulus E psi	Shear Modulus G psi	Bulk Modulus K psi
			DH 79-23 (Continued)				
152	18,750	3,614	0.481	166.2	1,386,000	468,000	11,977,000
151	18,750	3,297	0.484	166.2	1,156,000	390,000	12,082,000
150	18,750	3,529	0.482	166.2	1,323,000	446,000	12,006,000
149	15,000	3,750	0.467	156.7	1,394,000	475,000	6,970,000
148	13,636	3,947	0.454	152.0	1,485,000	511,000	5,414,000
147	10,714	3,797	0.428	137.9	1,225,000	429,000	2,842,000
146	11,538	3,797	0.439	142.6	1,276,000	443,000	3,503,000
145	11,538	4,280	0.420	142.6	1,600,000	563,000	3,343,000
144	9,375	3,898	0.395	128.4	1,174,000	421,000	1,873,000
143	14,286	5,527	0.412	154.4	2,873,000	1,017,000	5,440,000
142	10,000	5,587	0.273	133.2	2,283,000	897,000	1,678,000
141	8,571	3,499	0.4	121.4	898,000	321,000	1,496,000
140	10,345	4,223	0.4	135.5	1,459,000	521,000	2,433,000
139	7,895	3,223	0.4	114.3	717,000	256,000	1,195,000
138	6,383	2,606	0.4	93.1	382,000	136,000	636,000
137	6,667	2,722	0.4	97.8	438,000	156,000	729,000
136	6,818	2,783	0.4	100.1	468,000	167,000	781,000
135	6,000	2,449	0.4	86.0	311,000	111,000	519,000
134	5,556	4,556	--	76.6	325,000	343,000	53,000
133	6,250	2,552	0.4	90.7	357,000	127,000	594,000
132	6,000	4,754	--	86.0	551,000	419,000	109,000
131	6,977	2,848	0.4	102.5	502,000	179,000	837,000
130	6,000	2.449	0.4	86.0	311,000	111,000	519,000
129	6,383	4,510	--	93.1	818,000	408,000	274,000
128	10,000	4,379	0.381	133.2	1,522,000	551,000	2,138,000
127	8,571	4,409	0.320	121.4	1,344,000	509,000	1,245,000
126	8,571	3,499	0.4	121.4	898,000	321,000	1,496,000
125	7,143	2,916	0.4	104.9	539,000	192,000	898,000
124	8,108	4,401	0.291	116.6	1,258,000	487,000	1,004,000
123	12,500	5,103	0.4	147.3	2,316,000	827,000	3,861,000
122	15,000	6,124	0.4	156.7	3,549,000	1,267,000	5,914,000
121	16,667	6,804	0.4	161.4	4,515,000	1,612,000	7,525,000
120	16,667	4,354	0.463	161.5	1,932,000	660,000	8,795,000
119	14,286	3,659	0.465	154.4	1,306,000	446,000	6,202,000
118	17,647	3,846	0.475	163.8	1,542,000	523,000	10,304,000
117	15,789	3,750	0.470	159.1	1,419,000	483,000	7,910,000
116	13,636	3,846	0.457	152.0	1,413,000	485,000	5,449,000
115	11,538	4,813	0.395	142.6	1,987,000	712,000	3,144,000
114	10,345	4,223	0.4	135.5	1,459,000	521,000	2,433,000
113	9,677	3,951	0.4	130.8	1,233,000	440,000	2,054,000
112	8,333	3,402	0.4	119.0	832,000	297,000	1,386,000
111	8,824	3,602	0.4	123.7	967,000	346,000	1,616,000
110	12,000	4,403	0.422	144.9	1,723,000	606,000	3,692,000
109	18,750	4,964	0.462	166.2	2,581,000	8[illegible],000	11,424,000
108	23,077	9,421	0.4	173.2	9,283,000	3,315,000	15,472,000
107	14,460	4,676	0.442	155.0	2,107,000	731,000	6,015,000
106	20,000	8,165	0.4	168.5	6,783,000	2,423,000	11,306,000
105	15,789	4,412	0.458	159.1	1,947,000	668,000	7,663,000

(Sheet 3 of 3)

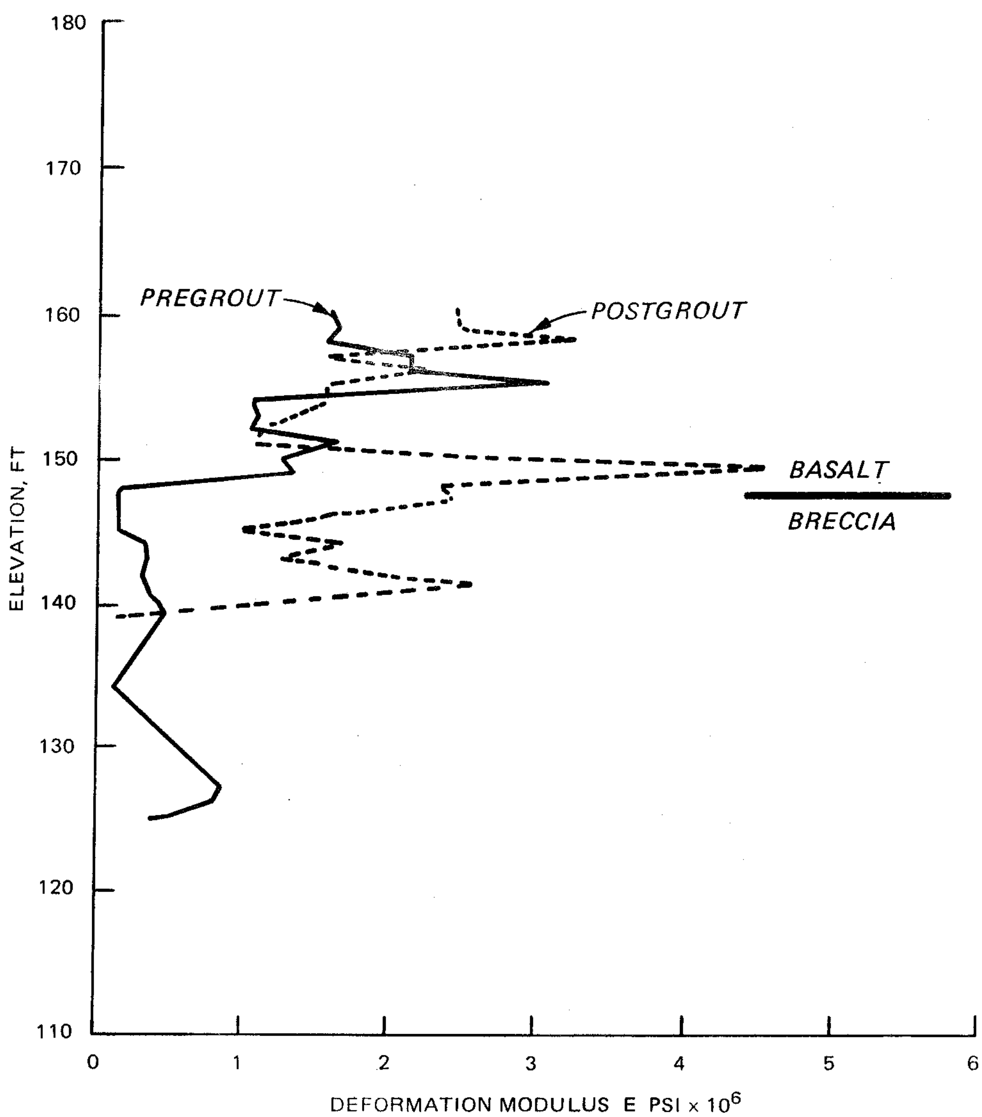


Figure 24. Comparison of moduli from 3-D logs, DH 79-21

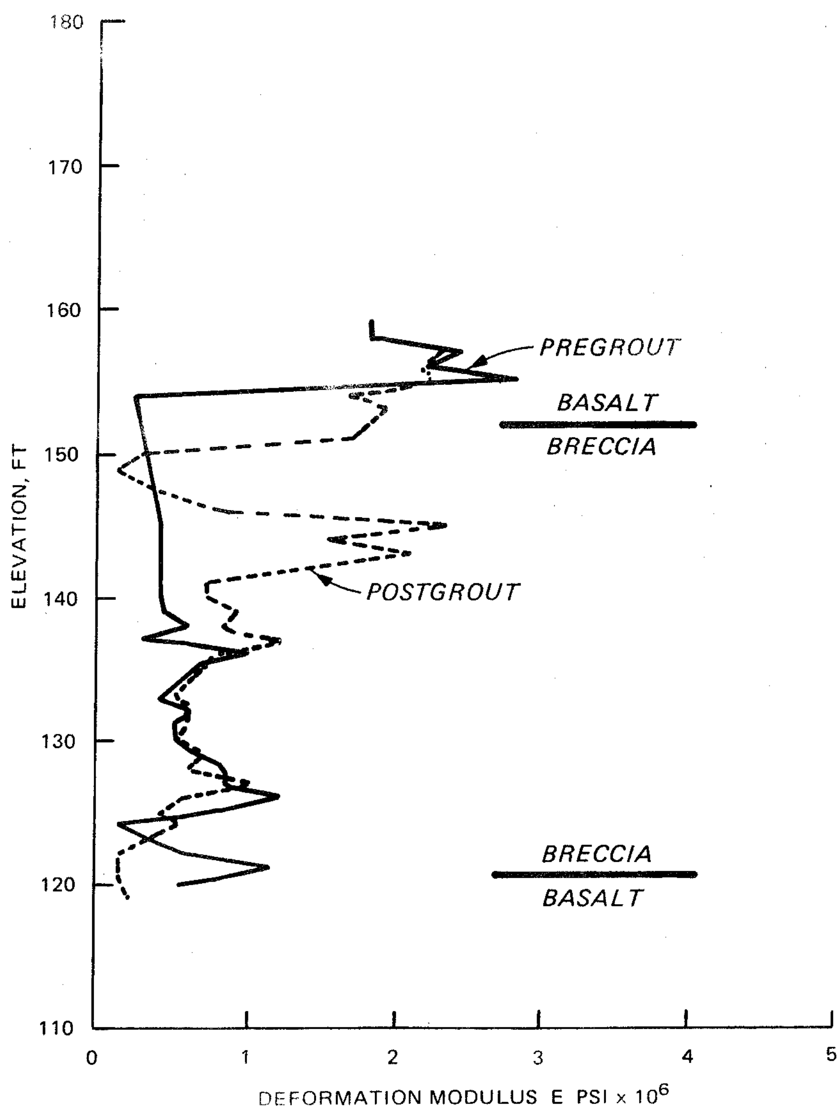


Figure 25. Comparison of moduli from 3-D logs, DH 79-22

140. The 3-D derived bulk densities were used in the data reduction to provide direct comparability with the pregrout results. The average density for basalt computed from the 3-D data (based on 72 measurements) was 150.2 lb/ft^3. The average density for grouted breccia computed from the 3-D data (based on 90 measurements) was 109.2 lb/ft^3. The postgrout average values for dynamic Young's, shear, and bulk moduli in all four holes are given.

Material	Young's Modulus psi	Shear Modulus psi	Bulk Modulus psi
Basalt (61 measurements)	2,141,000	759,000	6,987,000
Breccia (90 measurements)	1,015,000	381,00	1,618,000

141. The crosshole seismic tests were performed and interpreted for the postgrout survey the same as were reported for the pregrout survey. The initial plan was to place shot and receivers at the same elevations and, after a successful record was obtained, raise the entire array in 5-ft increments for successive shots until the energy source cleared the water in the hole preventing satisfactory energy coupling. Some difficulties were encountered in carrying out this plan because of caving in some of the boreholes. Modifications to the plan were made so that adequate data could still be obtained.

142. The crosshole seismic velocities (refracted and direct) and the computed dynamic elastic parameters are presented in Table 11 and are also shown in Figures 20-23. The bulk densities were not computed from the crosshole seismic data, rather, values of 150.2 lb/ft^3 for basalt and 109.2 lb/ft^3 for breccia were used, as obtained from the 3-D log results. The results of all usable crosshole data produced these averages:

Material	Young's Modulus psi	Shear Modulus psi	Bulk Modulus psi
Basalt (13 measurements)	2,355,000	838,000	5,641,000
Breccia (17 measurements)	1,138,000	453,000	1,154,000

143. The postgrout survey included a series of water injection tests, referred to as Ludgeon tests. Inflatable packers were used to isolate the

Table 11

Crosshole Seismic Results, Postgrout Survey

Shot Hole	Receiver Hole	Elevation ft	P-Wave Velocity V_p ft/sec	S-Wave Velocity V_s ft/sec	Poisson's Ratio ν	Young's Modulus E psi	Shear Modulus G psi	Bulk Modulus K psi
79-20A	79-23	158	13,286	6,200	0.361	3,389,000	1,245,000	4,058,000
		153	11,625	5,471	0.358	2,633,000	970,000	3,085,000
		148	11,625	5,167	0.377	2,382,000	865,000	3,225,000
		143	7,750	3,577	0.365	822,000	301,000	1,013,000
		138	7,750	3,875	0.333	943,000	354,000	943,000
		135	7,933	3,400	0.387	755,000	272,000	1,119,000
		130	7,933	3,400	0.387	755,000	272,000	1,119,000
		128	9,127	3,862	--	--	--	--
		127	10,400	4,333	--	--	--	--
79-23	79-21	157	17,143	6,667	0.411	4,063,000	1,440,000	7,600,000
		152	15,000	6,124	0.4	3,401,000	1,215,000	5,669,000
		147	10,000	4,082	0.4	1,340,000	479,000	2,234,000
		142	8,571	4,615	0.296	1,300,000	502,000	1,061,000
		139	7,500	4,000	0.301	981,000	377,000	822,000
		136	7,750	3,164	0.4	660,000	236,000	1,100,000
		134	8,200	3,348	0.4	739,000	264,000	1,232,000
		132	9,971	4,653	--	--	--	--
		129	10,800	--	--	--	--	--
		128	10,000	--	--	--	--	--
79-21	79-23	162	13,333	5,443	0.4	2,687,000*	960,000*	4,479,000
		157	17,143	4,138	0.469	1,630,000	555,000	8,780,000
		152	15,000	3,750	0.467	1,336,000	456,000	6,681,000
		147	12,000	3,750	0.446	1,317,000	456,000	4,057,000
		142	7,500	4,286	0.258	1,089,000	433,000	748,000
		137	8,882	6,040	0.070	1,838,000	859,000	712,000
79-21	79-22	163	12,714	3,560	0.457	1,197,000	411,000	4,689,000
		158	17,800	3,069	0.485	906,000	306,000	9,857,000
		153	14,833	6,056	0.4	3,326,000	1,188,000	5,543,000
		148	8,900	3,708	0.395	903,000	324,000	1,434,000
		143	8,900	3,708	0.395	903,000	324,000	1,434,000
		138	9,889	4,037	0.4	1,075,000	384,000	1,791,000
79-20A	79-22	158	14,381	5,033	0.430	2,347,000	821,000	5,605,000
		148	9,438	7,190	--	--	--	--
		143	8,882	6,040	0.070	1,838,000	859,000	712,000
		138	8,882	6,040	0.070	1,838,000	859,000	712,000
		133	9,742	5.033	0.318	1,572,000	597,000	1,440,000
		128	10,786	6,565	--	--	--	--
		123	12,583	7,947	--	--	--	--
		121	11,615	8,389	--	--	--	--

* Poisson's Ratio assumed to equal to 0.4 when available data precluded calculation.

desired reach of a borehole. Water was injected into the zone and injection rate and applied pressure were monitored simultaneously. The test was to provide a characterization of the rock zone in terms of discontinuity frequency and dimensions. In this application, the grouted breccia was to be tested to determine the blockage of preexisting interstices by grout. In preparing equipment and the survey plan, it was assumed that most, if not all, interstices would be blocked by grout, resulting in very low flow rates. Further, injection pressures were to be held below the overburden pressure of the rock above any tested zone to maintain mechanical integrity of the grouted foundation. Consequently, rather than apply constant high supply heads and monitor flow rates, it was decided to use a falling-head type of test.

144. A 5-ft zone at the bottom of DH 79-22 was sealed off with a 10-ft-long rubberized packer inflated to 150-psi pressure. A 1-in.-ID pipe extended from the test zone, through the packer, to 5 ft above ground surface. A pressure transducer (maximum: 25 psi; resolution: 0.008-ft head of water) was lowered through the pipe into the test zone. The standpipe was to be filled with water, the water source removed, and the water allowed to drop naturally in the standpipe while measuring the water column height versus time. The data would supply both rate of flow and pressure head instantaneously.

145. After water was run into the standpipe at the maximum flow obtainable for 30 to 45 min, the water level had risen only 3.86 ft above its original level and could not be further increased. The water supply was removed and the falling water column was timed by stopwatch as the head was measured with the pressure transducer. Less than 2 sec was required for the water column to drop to its original level. This procedure was repeated twice with similar results. The packer and standpipe were removed leaving only the pressure transducer in the hole. Water was again piped into the open hole as fast as possible. Even less head increase could be obtained, probably because even more open interstices were available to outflow. The effort was abandoned. No usable quantitative data were provided by the falling-head test.

146. The averages of the deformation modulus (static, localized test type) determined by the Goodman borehole jack after and before grouting are these.

Test Hole	Modulus E , psi Basalt	Breccia
	After Grouting	
DH 79-20A	425,000-1,512,600	49,000-197,600
DH 79-23	453,400-1,548,100	130,400-309,300
DH 79-21	854,600-2,830,000	--
DH 79-22	1,541,900-4,973,300	85,100-331,680
	Before Grouting	
DH 79-21	--	43,800-179,200
DH 79-22	1,180,000-4,660,000	118,800-467,000

There is little discernible difference in the moduli, as measured by the Goodman borehole jack, between pregrout and postgrout survey results. This small variation in moduli may be a real lack of change in the elastic parameters or it may be a result of the application of the jack in very irregular holes with many loose fragments in the walls of the boreholes. To examine the test results more critically, a test rejection criterion was determined. The lowest hydraulic pressure achieved in the reported pregrout tests, before platen cocking occurred, was 800 psi. If a peak pressure of 800 psi or less were used as a criterion for discarding tests from the postgrout survey, then six tests quoted herein would be discarded, all from the breccia zone and all demonstrating severe platen cocking leaving these average moduli values:

Test Hole	Number of Tests Remaining	Modulus E , psi, Breccia
DH 79-20A	3	71,300-280,100
DH 79-23	3	131,900-464,200
DH 79-21	0	--
DH 79-22	3	130,800-502,700
Average	9	111,300-415,700

The values thus selectively chosen and averaged show a 10.1 percent increase on the lower end of the range and 7.7 percent increase on the upper end of the range when compared with the pregrout Goodman jack results from breccia in DH 79-22.

147. The averages of the dynamic moduli (dynamic, localized test type) determined by the 3-D sonic logs after grouting are these:

Test Hole	Young's Modulus E_d, psi			
	Basalt		Breccia	
	Pregrout	Postgrout	Pregrout	Postgrout
DH 79-20A	--	1,896,000	--	1,119,000
DH 79-23	--	2,315,000	--	1,098,000
DH 79-21	1,626,000	2,166,000	371,000	1,359,000
DH 79-22	2,241,000	1,992,000	626,000	750,000

A comparison of postgrout and pregrout results from DH 79-21 and DH 79-22 shows the following variations of Young's modulus in the basalt and breccia.

Test Hole	Variation, %	
	Basalt	Breccia
DH 79-21	+33	+266
DH 79-22	-11	+ 20

The modulus of the breccia in DH 79-22 shows a lower degree of improvement, but still it is an appreciable factor. The breccia moduli from all four holes after grouting are within a standard deviation of ±217,000 psi from the average value of 1,082,000 psi.

148. The crosshole seismic results (dynamic, large volume test type) for dynamic moduli after grouting are given.

Crosshole (From-To)	Young's Modulus E_d, psi			
	Basalt		Breccia	
	Pregrout	Postgrout	Pregrout	Postgrout
DH 79-20A to DH 79-23	--	2,801,000	--	819,000
DH 79-23 to DH 79-21	2,150,000	2,406,000	514,000	1,135,000
DH 79-21 to DH 79-22	1,470,000	1,810,000	498,000	960,000

The increases in breccia moduli are appreciable. A comparison of crosshole seismic results and 3-D log results in terms of the dynamic Young's modulus of breccia is shown. The results of the two methods are comparable in magnitude and also in the trend toward a lower postgrout modulus in the region near DH 79-22. The low crosshole modulus measured between DH 79-20A and 79-23 is anomalous when compared with the other two transmission lines and when compared with the values from the two adjacent boreholes.

Borehole	3-D Modulus, psi	Borehole	Crosshole Seismic, Modulus psi
DH 79-20A	1,119,000	DH 79-20A	819,000
		to	
DH 79-23	1,098,000	DH 79-23	
		to	1,135,000
DH 79-21	1,359,000	DH 79-21	
		to	960,000
DH 79-22	750,000	DH 79-22	

149. The postgrout deformation moduli from the borehole jack, the 3-D sonic log, and the crosshole seismic survey are plotted versus depth in Figures 20-23. Pregrout and postgrout results of the 3-D log modulus measurements are compared in Figures 24 and 25.

150. Some of the anomalous test results and high water takes experienced in the water injection tests can be explained by the survey hole locations. The relatively small area between the lock wall and the fishladder controlled the locations for survey hole drilling. The accessible locations, as shown in Figure 15, are on the outermost row of groutholes. The outermost row was grouted first to confine grout travel for subsequent grouting operations. A well performed grouting job would have better void filling nearer the lock wall. In reviewing the survey plan, an improved pregrout-postgrout characterization would be possible if tests were made in central grouted areas.

151. In future comparison surveys of this type, the borings used for postgrout measurement should be freshly drilled. The action of backfilling with sand and subsequent cleaning-out degraded the already poor wall conditions in the fragile breccia. It would be inappropriate to grout the pregrout holes and later rebore them as that would give measurements weighted in favor of anomalous grout concentrations. Offset postgrout borings should be used.

152. The combination of techniques used to obtain dynamic moduli, the 3-D log and the crosshole seismic test, is excellent. The 3-D sonic results are localized near the boreholes but are extremely redundant in depth; whereas, the crosshole seismic results are generalized from large volumes or rock but are essentially very localized in terms of depth. Any comparison or evaluation survey suite in the future should include both of these techniques.

153. Considerations concerning the effectiveness of the foundation grouting program include a comparison of expected and actual deflection

reductions, pregrout and postgrout modulus measurements, along with a review of expected and actual grout takes.

154. Previous grouting at this site and field permeability testing were used to make grout take estimates. Actual grout take totaled approximately 33,180 ft^3 (sacks) of cement--the same order of magnitude as estimated, although variations did occur in takes within the two primary rock types and along the axis of the lock. The fact that maximum grout takes occurred in the flow breccia adjacent the middle monoliths, where maximum deformations have occurred, emphasized the need for grouting to fill voids and fractures within the foundation rock.

155. Pregrout and postgrout modulus determinations showed a general increase in the in situ modulus of the flow breccia from about 500,000 psi to about 1,000,000 psi, with good agreement between the 3-D acoustic velocity measurements and the crosshole seismic tests. However, the in situ modulus determinations were made along the exterior grout line where pressures were kept low to discourage grout travel, and the results of those in situ measurements may not be representative of the grouted rock mass.

156. The prediction of monolith deflection reductions as a result of foundation grouting came primarily from finite element studies of rock/structure interaction. These studies were based on rather sparse pregrout field measurements (1979 and earlier) as well as a number of assumptions regarding stress distribution, variations in rock moduli, effects of the crack, and the possible separate effects of each portion of the total repair scheme. A wire line system, which measures total monolith movements, showed deflection reductions in excess of 50 percent from the projected pregrout conditions. Figure 26 shows a plot of the pregrout and postgrout deflections data versus time and temperature. The reduction in deflection represents very strong evidence of grout effectiveness considering that the reductions resulted almost exclusively from changes in the translation mode of lock wall movement. The bar extensometer data showed a reduction in monolith deflections resulting from grouting, although only in the range of 20 to 30 percent from pregrout movements. Because the bar extensometer measures only relative movement between monoliths, the noted reduction indicated an improvement but did not completely explain the results, since bar extensometer measurements include translation, rotation, and flexure components.

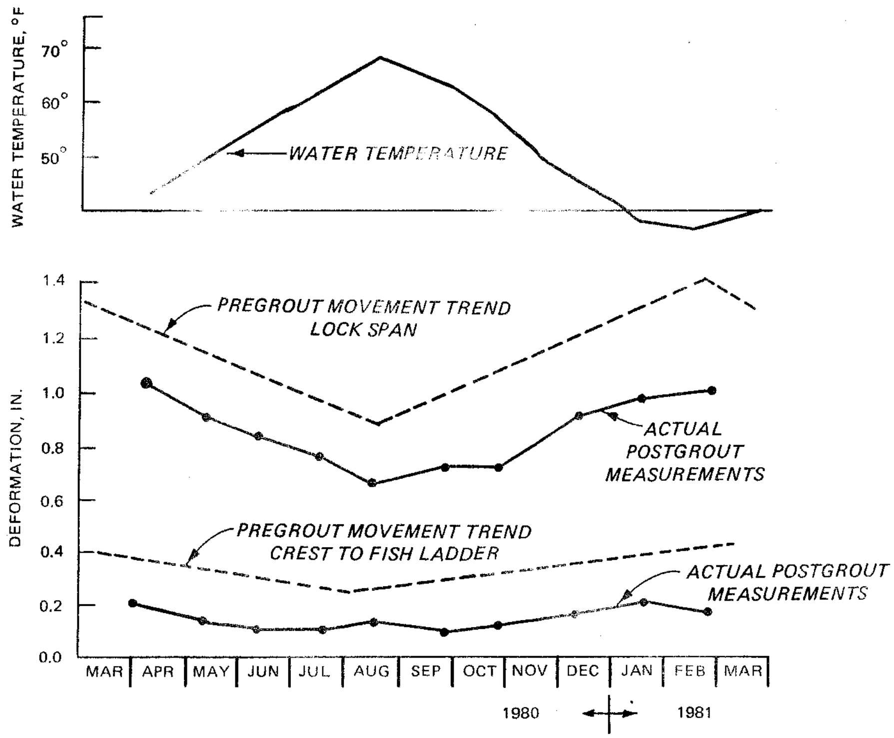


Figure 26. Pregrout and postgrout wire line data, monolith deflection (after Neff, Sager, and Griffiths 1982)

157. In summary, the foundation grouting achieved the objective intended, although deflection did not exactly match expectations (50 percent actual reduction versus 70 percent expected). An additional bonus resulting from the foundation grouting program was the obtaining of a better understanding of the complex nature of the monolith deflections, especially the contribution to monolith rotation caused by the presence of a structural crack, which greatly weakened the overall flexural rigidity of the lock wall/ foundation (Neff, Sager, and Griffiths 1982).

PART VI: CONCLUSIONS AND RECOMMENDATIONS

Summary and Findings

158. The groutability of a rock mass, or the ability of a rock mass to accept grout, is dependent on site-specific geological features. The features include the geometrical aspects of the discontinuities into which the grout is injected, such as spacing, width, length, orientation, and surface roughness. Standard geological mapping procedures combined with statistical analysis appear to be the best method of characterizing the discontinuities in a rock mass.

159. Details of various grouting methods are fully documented elsewhere, e.g. US Army Engineers 1984. Considerations affecting the quality of the grouting job include grout hole spacing and diameter, drilling and washing techniques, applied grouting pressures, and the grouting equipment. Grout hole spacing, diameter, and drilling methods are largely dependent on the site-specific geological conditions. In CE applications in the past, grouting pressures have been limited to 1 psi/ft of depth. Under certain conditions, higher pressures can be applied without damaging the rock mass or adjacent structures.

160. Current monitoring and evaluation techniques are generally based on an analysis of grout takes and water takes, as recorded by the grouting inspector. Adoption of electronic monitoring devices will allow collection of highly accurate grouting data.

161. Grouting evaluation methods have ranged from simple observations of structure performance to indirect geophysical testing techniques. Evaluations based on unit grout takes and observation of grouting techniques have the advantage of analyzing job quality as the job is performed. Operational adjustments can be made immediately. Methods relying on core sampling or geophysical testing typically are completed subsequent to grouting. If deficiencies in the job quality are found, the contractor would have to remobilize.

162. Only three case histories of remedial consolidation grouting were identified within the CE. The need for remedial grouting was identified by excessive structure deflections for the John Day Lock and Dam and the Little Goose Lock and Dam. Remedial consolidation grouting was performed

under the Savage River Dam spillway after an increased flood flow was identified as a design requirement. In each of these three cases, standard grouting techniques, cement mixes, and monitoring methods were implemented for the respective grouting programs. Geophysical testing performed at the John Day Lock and Dam showed the grouting program met design objectives.

163. The Little Goose Lock and Dam had two major problems that engineers hoped grouting would cure. One was permanent outward deflection of a lock wall. The second was temporary differential movements during filling of the lock. No information was available to the author to verify the present status of the dam and the grouting effectiveness. The grouting was designed and placed after test grouting.

164. The Savage River Dam spillway rehabilitation was completed and was considered to be effectively grouted over 90 percent of the area to a depth of 10 ft below the spillway floor. However, this conclusion was supported only by observations during grouting based on grout take. Structural performance appears to be adequate, although the spillway has not been subjected to design flood.

165. The John Day Lock and Dam had structural cracking caused by differential movements during lock filling. The foundation grouting achieved the objectives intended, although deflection reduction did not exactly match expectations (50 percent actual reduction versus 70 percent expected); however, the complex nature of the monolith's deflection was better understood, especially the contribution of monolith rotation caused by the presence of the structural cracks. A small change in dynamic modulus can have a great effect on the total movement of a structure.

Recommendations

166. This study showed areas where future studies should be directed. Recommendations for future work are presented.

167. Properties of cement grouts, at the time of mixing or after setting, are measured. Likewise, geological features can be identified easily. However, the two facets of grouting need to be combined. The flow of cementitious grouts through real as opposed to idealized rock mass discontinuities should be studied.

168. Evaluation of a grouting job typically consists of indirect methods during grouting (i.e., measuring grout takes) or direct measurements obtained after completion of grouting (i.e., geophysical techniques and core sampling). Development of a grout monitoring system that could determine grout location during injection would be beneficial. Acoustic emission techniques is one promising technology for providing real-time grout location.

169. The geological descriptions of the strata encountered in the drill holes were complete for each of the case histories. Extensive laboratory testing had been performed to obtain design parameters. However, RQD values and angles of natural fractures were not recorded on the available drill logs. Analyses of discontinuity patterns, including standard items such as stereonet plots, were not presented. This lack of data precludes thorough analyses of grouting programs.

170. Improved rock mass characterizations may be possible through statistical methods. As familiarity with statistical methods increases, such characterizations will allow improvements in grout program designs and implementation.

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